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Development of Risk Assessment Methodology for Surface Disposal of Municipal Sludge

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This is one of a series of reports that present methodologies for assessing the potential risks to humans or other organisms from the disposal or reuse of municipal sludge. The sludge management practices addressed by this series include land application practices, distribution and marketing programs, landfilling, surface disposal, incineration and ocean disposal. In particular, these reports provide methods for evaluating potential health and environmental risks from toxic chemicals that may be present in sludge. This document addresses risks from chemicals associated with surface disposal of municipal sludge. These proposed risk assessment procedures are designed as tools to assist in the development of regulations for sludge management practices. The procedures are structured to allow calculation of technical criteria for sludge disposal/reuse options based on the potential for adverse health or environmental impacts. The criteria may address management practices (such as site design or process control specifications), limits on sludge disposal rates or limits on toxic chemical concentrations in the sludge.

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DEVELOPMENT OF RISK ASSESSMENT METHODOLOGY FOR SURFACE DISPOSAL OF MUNICIPAL SLUDGE

Environmental Criteria and Assessment Office
Office of Health and Environmental Assessment
Office of Research and Development
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PREFACE

This is one of a series of reports that present methodologies for assessing the potential risks to humans or other organisms from the disposal or reuse of municipal sludge. The sludge management practices addressed by this series include land application practices, distribution and marketing programs, landfilling, surface disposal, incineration and ocean disposal. In particular, these reports provide methods for evaluating potential health and environmental risks from toxic chemicals that may be present in sludge. This document addresses risks from chemicals associated with surface disposal of municipal sludge.

These proposed risk assessment procedures are designed as tools to assist in the development of regulations for sludge management practices. The procedures are structured to allow calculation of technical criteria for sludge disposal/reuse options based on the potential for adverse health or environmental impacts. The criteria may address management practices (such as site design or process control specifications), limits on sludge disposal rates or limits on toxic chemical concentrations in the sludge.

The methods for criteria derivation presented in this report have been submitted to the U.S. EPA Office of Water Regulations and Standards (OWRS) to provide scientific background for the development of technical criteria for toxic chemicals in sludge. However, the methods used by OWRS to develop regulations could differ from this guidance in some respects; therefore, the Technical Support Documents provided by OWRS should be consulted for explanation of the regulations.

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LIST OF ABBREVIATIONS

Γ	Cumulative transport factor for cascade model (unitless)
γ	Transport factor for cell in cascade model (unitless)
Θ	Temperature correction factor (unitless)
σ_z	Standard deviation of contaminant plume above the ground (m)
α_L	Longitudinal dispersivity (m)
η	Effective water storage capacity (m^{-1})
ϕ	Effective porosity (unitless)
Λ	First-order contaminant loss due to atmospheric decay and deposition (unitless)
λ	First-order decay constant (sec^{-1})
μ_a	Viscosity of air (g/cm-sec)
μ_w	Viscosity of water (g/cm-sec)
ψ	Pressure head (m)
ψ_a	Air entry pressure head (m)
ρ_b	Bulk density of the wet soil (g/m^3)
ρ_a	Air density (g/m^3)
ρ_w	Water density (g/m^3)
θ	Volumetric water content (unitless)
7Q10	Minimum average stream flow expected to occur once every 10 years (m^3/sec)
A	Area of air-exposed surface of water (m^2)
AC	Upper-bound estimate of contaminant concentration in air at property boundary (g/m^3)
AWQC	Ambient Water Quality Criteria
B	Width of cell in surface water model (m)
BCF	Unadjusted bioconcentration factor in fish (*l*/kg)
BCF_a	Adjusted bioconcentration factor (*l*/kg)

BI	Background intake of pollutant from a given exposure route (mg/day)
BW	Human body weight (kg)
C	Contaminant concentration (g/m^3)
c	Concentration of solute in unsaturated zone (g/m^3)
C_b	Background concentration of contaminant in the environmental medium (mg/*l* or mg/m^3)
C_d	Concentration (dissolved form) (g/m^3)
C_E	Concentration of contaminant in effluent (g/m^3)
C_f	Fluid compressibility ($m\text{-}sec^2/g$)
C_g	Concentration in groundwater recharged to river (g/m^3)
C_{gw}	Groundwater concentration of contaminant (mg/*l*)
C_l	Contaminant concentration in liquid (g/m^3)
C_{max}	Maximum allowable contaminant concentration in impoundment (mg/*l*)
C_0	Inlet contaminant concentration (g/m^3)
C_p	Concentration (particulate form) (g/m^3)
C_s	Contaminant concentration in solid (g/kg)
C_u	Concentration in river upstream of sludge disposal site (g/m^3)
d	Aquifer depth (m)
D	Dispersion coefficient (m^2/sec)
D^*	Effective molecular diffusion coefficient (m^2/sec)
$\{D\}$	Dispersion tensor (m^2/sec)
D_{ca}	Diffusivity of contaminant in air (cm^2/sec)
D_{af}	Anti-dilution factor (unitless)
d_e	Effective distance (m)
D_{eth}	Diffusivity of ether in water (cm^2/sec)
D_f	Dilution factor (unitless)
D_{cw}	Diffusivity of contaminant in water (cm^2/sec)
E_{max}	Maximum allowable air emission of contaminant (g/sec)

E_{site}	Emission rate of contaminant to air from site surface (g/sec)
F_a	Volume of fluid passing through a vertical cross-section of an aquifer oriented perpendicular to the direction of flow (m^3/sec)
FD	Fetch-to-depth ratio (unitless)
f_d	Dissolved fraction of contaminant (unitless)
f_{ij}	Frequency of specific stability array parameters for class indication i,j (stability class, windspeed)
f_{oc}	Weight fraction of organic carbon in total solids (unitless)
F_s	Volume of fluid leaving surface impoundment (m^3/sec)
g	Gravitational acceleration (m/sec^2)
GEMS	Graphical Exposure Modeling System
H	Mean flow depth (m)
H_c	Henry's law constant for the contaminant (atm-m^3/mol)
H_e	Dimensionless Henry's law constant (unitless)
HE_{TOT}	Total human exposure to contaminant (mg-day/kg)
HHAG	Human Health Assessment Group
h	Impoundment depth (m)
h_e	Effective height (m)
I_a	Total air inhalation rate (m^3/day)
ICRP	International Commissions on Radiological Protection
I_f	Total fish ingestion rate (kg/day)
ISCLT	Industrial Source Complex Long-Term
I_w	Total water ingestion rate (day^{-1})
K	Vertical hydraulic conductivity (m/sec)
k	Overall mass transfer coefficient for volatilization (m/sec)
k_{agg}	"Aggregate" decay rate, or the arithmetic sum of k_p, k_h and k_b (sec^{-1})
k_b	First-order biodegradation constant (sec^{-1})
k_d	Equilibrium distribution coefficient for concentration (*l*/kg)
K_d	Loss rate due to contaminant decay (m/sec)

k_g	Gas phase mass transfer coefficient (m/sec)
k_h	Loss rate due to hydrolysis (sec^{-1})
k_l	Liquid phase mass transfer coefficient (m/sec)
K_{ow}	Octanol-water partition coefficient (unitless)
k_p	Loss rate due to photolysis (sec^{-1})
k_{rw}	Effective permeability (unitless)
k_s	Loss rate due to interaction with sediment (sec^{-1})
k_{sl}	Equilibrium distribution coefficient for mass (unitless)
K_{tot}	Net contaminant loss rate (m/sec)
L	Cell length (m)
LC_d	Lipid content of dietary seafood (kg/kg)
LC_e	Lipid content of experimental organism (kg/kg)
L_{GW}	Contaminant loadings to groundwater (g/sec)
L_m	Longitudinal mixing distance (m)
M	Mass flux (g/m^2-sec)
m	Mass of contaminant (g)
M_B	Mass of contaminant removed by biodegradation (g/sec)
MCLG	Maximum Contaminant Level Goal
M_E	Mass of contaminant removed by effluent or seepage (g/sec)
MEI	Most exposed individual
MEP	Most exposed population
MEU	Most exposed unit
MGD	Million gallons per day
M_I	Mass of contaminant entering the facility (g/sec)
M_o	Mass of contaminant leaving facility (g/sec)
M_{SL}	Mass transfer between solid and liquid phases (g)
ΔM_T	Change in total contaminant mass within the facility (g/sec)
M_{VA}	Mass of contaminant removed by volatilization due to wind (g/sec)

M_{VD}	Mass of contaminant removed by volatilization of resulting from diffused air (g/sec)
N_i	Total concentration of contaminant i in sludge (mg/kg or g/Mg, dry weight)
N_{max}	Maximum allowable concentration of contaminant in sludge (mg/kg or g/Mg, dry weight)
n	Manning's roughness coefficient (unitless)
NRC	National Research Council
OHEA	Office of Health and Environmental Assessment
OWRS	Office of Water Regulations and Standards
Q	Liquid volumetric flow rate (m^3/sec)
Q_a	Volumetric flow rate of diffused air (m^3/sec)
q_d	Discharge per unit width (m^2/sec)
Q_f	Mean annual flow (m^3/sec)
Q_g	Groundwater discharge to river (m^3/sec)
q_i	Infiltration rate from impoundment (m/sec)
Q_s	Quantity of sludge entering facility (Mg/year, dry weight)
Q_u	Upstream discharge (m^3/sec)
q_1^*	Human cancer potency ((mg-day/kg)$^{-1)}$)
r	Radius of circle with same area as impoundment (m)
R	Ideal gas constant = 8.206×10^{-5} (m^3-atm/mol-K)
RF	Retardation factor in groundwater (unitless)
RAC	Reference air concentration (mg/m^3)
RfD	Reference Dose (mg/day-kg)
RE	Relative effectiveness of exposure route (unitless)
RL	Risk level (unitless)
r_{sl}	Ratio of total solids to liquid in flow (kg/*l*)
RWC	Reference water concentration (mg/$_l$)
S	Longitudinal slope of the cell (unitless)
Sc_g	Schmidt number on gas side (unitless)

Sc_l	Schmidt number on liquid side (unitless)
S_e	Effective water saturation (unitless)
S_s	Specific storage (unitless)
S_w	Water saturation (unitless)
S_{wr}	Residual water saturation (unitless)
SRR_{GW}	Source-receptor ratio for groundwater (kg/*l*)
SRR_{SW}	Source-receptor ratio for surface water (kg/*l*)
SRR_{VOL}	Source-receptor ratio for volatilization (kg/m^3)
STAR	Stability array
T	Temperature (°K)
t	Time (sec)
TBI	Total background intake rate of contaminant (mg/day) from all other sources of exposure
U_{10}	Mean windspeed at 10 meters above lagoon surface (m/sec)
U^*	Friction velocity
U_s	Mean flow velocity of water (m/sec)
U_w	Mean windspeed (m/sec)
V	Volume of liquid (m^3)
V_D	Darcy velocity (m/sec)
V_h	Regional velocity of horizontal groundwater flow (m/sec)
V_i	Increase in groundwater velocity due to the impoundment (m/sec)
V_T	Transport velocity (m/sec)
V_v	Vertical velocity due to the impoundment (m/sec)
W	Contaminant mass loading (g/sec)
w	Width of impoundment perpendicular to direction of flow (m)
X	Downstream distance (m)
X_a	Distance in the x-coordinate direction (parallel to wind velocity, u_{wj}) from source to point of interest (m)
X_i	Maximum expected dissolved concentration of contaminant i (mg/*l*)

x_{vi}	Virtual distance required for point source plume to spread to width of site (m)
X_{sw}	Distance from center of impoundment to surface water (m)
Y_a	Distance in the y-coordinate direction (perpendicular to wind velocity U_{wj}) from source to point of interest (m)
z	Vertical coordinate in the unsaturated zone (m)
Z_a	Distance in the z-coordinate direction (perpendicular to wind velocity, U_{wj}) from source to point of interest (m)

1. INTRODUCTION

1.1. PURPOSE AND SCOPE

This report is one in a series of documents that describe methodologies used to evaluate the potential health and environmental risks resulting from the management (i.e., disposal or reuse) of municipal sewage sludge, including application to land for beneficial purposes, landfilling, incineration and ocean dumping. This document addresses health and environmental risks associated with human and non-human exposure to chemical contaminants in sludge from the surface disposal of municipal sludge, and explains the development of a methodology by which criteria for sludge surface disposal may be derived.

The term "surface disposal" will refer in this document to the permanent disposal or long-term storage of sludge in uncovered lagoons or impoundments. Short-term storage of sludge during wastewater treatment (i.e., less than one year) is not included in this definition. The land application of sludge, the distribution and marketing of sludge and the disposal of sludge in covered landfills, are also excluded from this definition. Methodologies for deriving criteria for these sludge management practices are described by other documents in this series (U.S. EPA 1986a, 1989d). In addition, disposal or long term storage of sludge in waste piles, which might also be described appropriately as "surface disposal," is not included in the present discussion.

The risk evaluation methods described in this document are intended to aid in the development of regulations for the surface disposal of sludge. The procedures are structured to allow calculation of technical criteria for regulating sludge disposal/reuse, based on potential adverse health and environmental impacts. These criteria may include restrictions on the concentrations of chemical contaminants in sludge and restrictions governing the design and management of sludge disposal sites. Restrictions on concentrations of chemical contaminants consist of maximum allowable dry-weight concentrations for each contaminant in the sludge.

The methods for deriving criteria presented in this report are intended to be used by the Office of Water Regulations and Standards (OWRS) to develop national technical criteria for toxic chemicals in municipal sludge. This document is not intended to be a manual to guide users in estimating potential risks from a particular sludge management site. A user's manual based on the methods described in this document should be developed separately.

This report is part of a series of methodology background documents. This series does not address potential health risks from the presence of pathogenic organisms in municipal sludge, which are examined in separate U.S. EPA analyses. These documents also do not address health and ecological risks from the treatment, transport, handling or accidental release of sludge.

1.2. DEFINITIONS AND COMPONENTS OF RISK ASSESSMENT

The National Research Council (NRC) defines risk assessment as the process of characterizing the potential adverse human health or environmental effects of exposures to environmental hazards (NRC, 1983). Risk management, on the other hand, is defined as the process of evaluating alternative regulatory actions that may be taken to reduce risk and choosing among them, based on consideration of costs, availability of technologies, and other factors.

NRC defines four components of risk assessment. The first step is hazard identification. In this process, relevant data are gathered and assessed to evaluate whether exposure to a particular agent poses a health or environmental hazard. The next step, the dose-response assessment, is used to estimate the likely level of response observed given a particular level of exposure to a toxic agent. The evaluation of dose-response data involves quantitatively characterizing the relationship between the amount of exposure and extent of toxic injury or disease. The U.S. EPA has broadened the definitions of hazard identification and dose-response assessment to include the nature and severity of the toxic effect in addition to the incidence. Procedures for hazard identification and for developing dose-response assessments have been established by the Agency and are followed in this methodology document. Exposure assessment, the third step in risk assessment, is the process of measuring the intensity, frequency and duration of exposure to an agent currently present, or of estimating hypothetical exposures that might arise. Risk characterization, step four in risk assessment, is performed by combining the exposure and dose-response assessments to estimate the likelihood of an effect (NRC, 1983).

Figure 1-1 shows the relationship of the development of risk assessment methodologies to sludge management practices. The figure further shows how each method may be used to develop

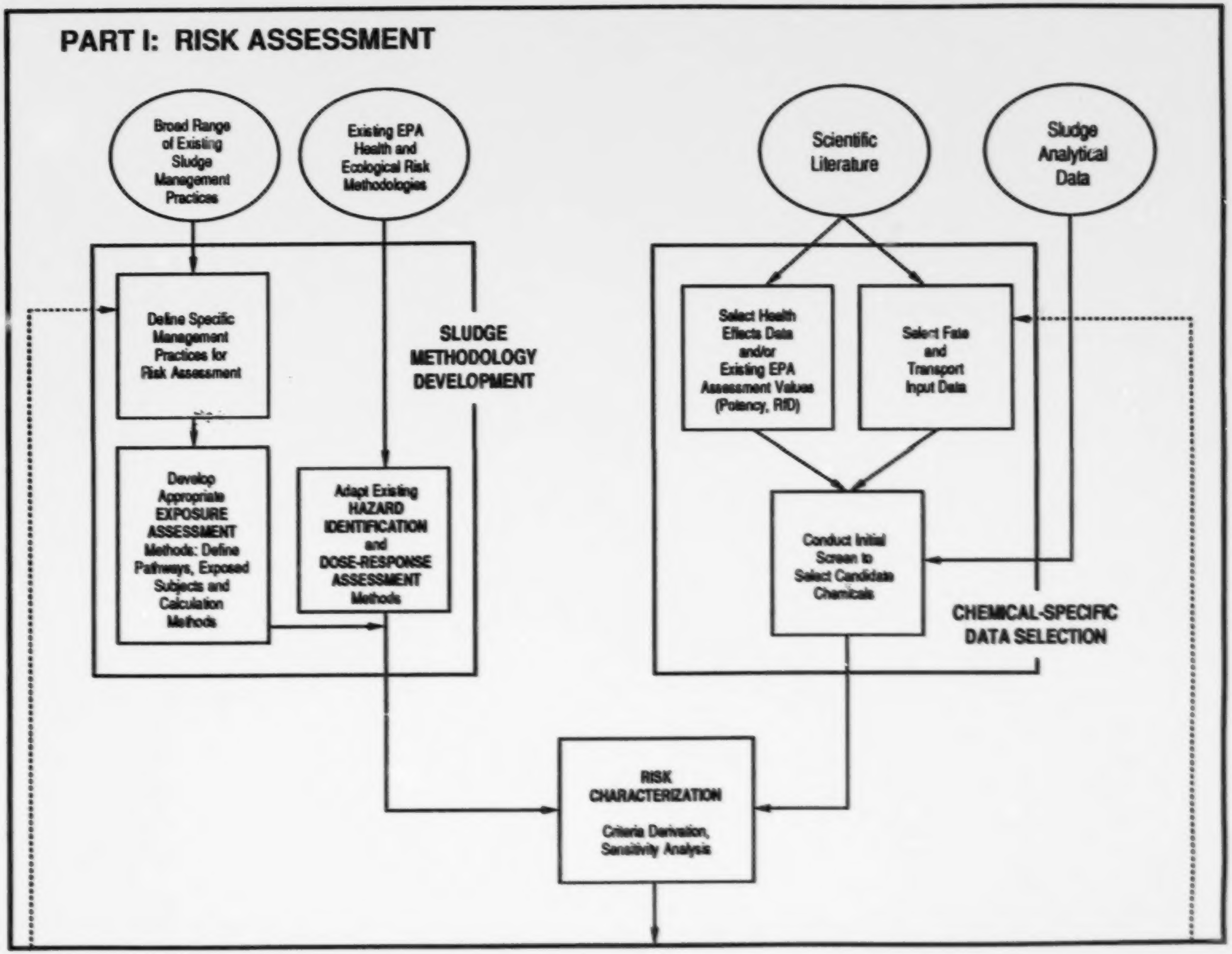
PART I: RISK ASSESSMENT
Broad Range of Existing Sludge Management Practices
Existing EPA Health and Ecological Risk Methodologies
Scientific Literature
Sludge Analytical Data
Define Specific Management Practices for Risk Assessment
SLUDGE METHODOLOGY DEVELOPMENT
Develop Appropriate EXPOSURE ASSESSMENT Methods: Define Pathways, Exposed Subjects and Calculation Methods
Adapt Existing HAZARD IDENTIFICATION and DOSE-RESPONSE ASSESSMENT Methods
Select Health Effects Data and/or Existing EPA Assessment Values (Potency, RfD)
Select Fate and Transport Input Data
Conduct Initial Screen to Select Candidate Chemicals
CHEMICAL-SPECIFIC DATA SELECTION
RISK CHARACTERIZATION
Criteria Derivation, Sensitivity Analysis

FIGURE 1-1

Relationship of Risk Assessment Methodology to Other Components of Regulation Development for Sewage Sludge Reuse/Disposal Options

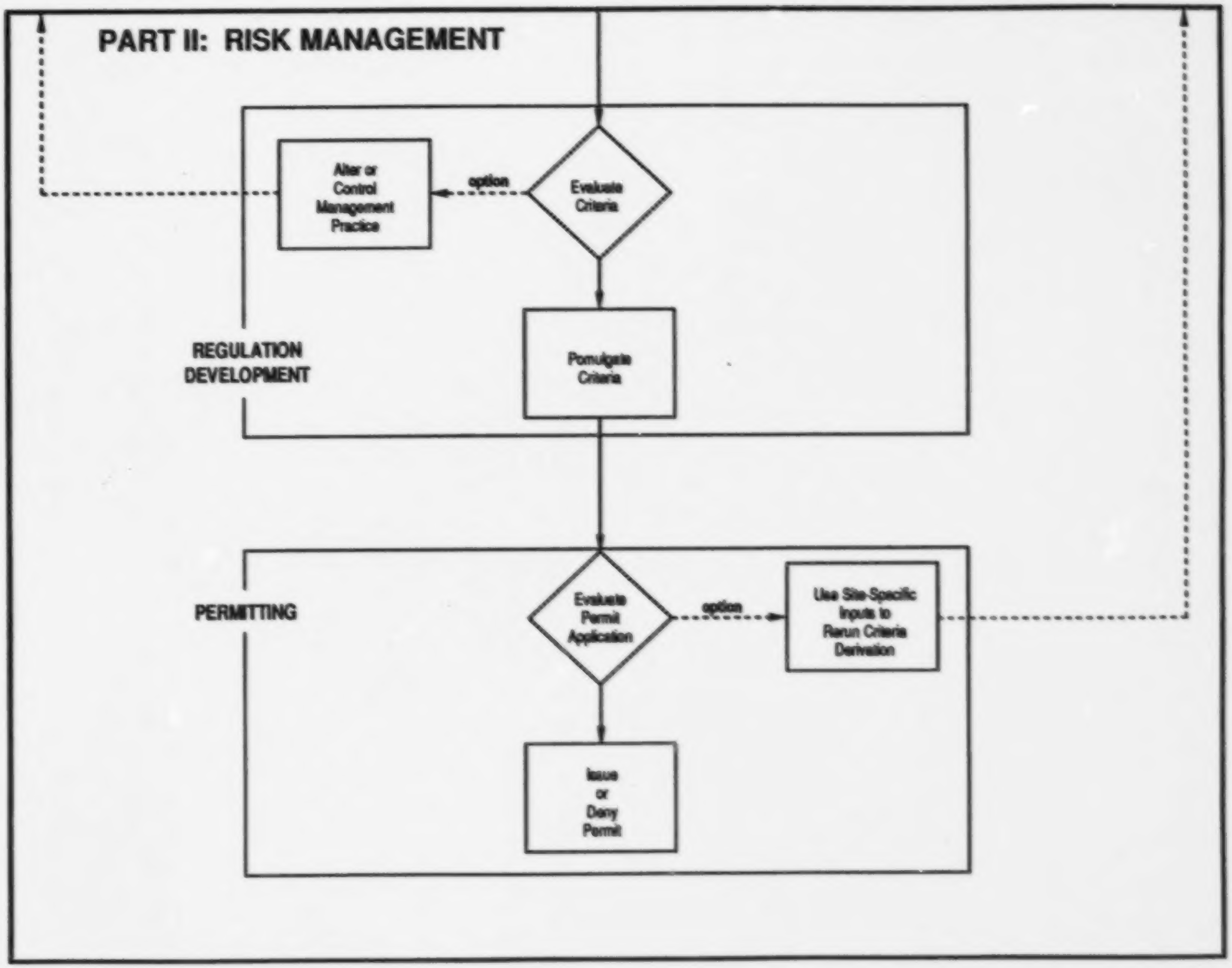
PART II: RISK MANAGEMENT
REGULATION DEVELOPMENT
Alter or Control Management Practice
option
Evaluate Criteria
Pomulgate Criteria
PERMITTING
Evaluate Permit Application
option
Use Site-Specific Inputs to Rerun Criteria Derivation
Issue or Deny Permit

FIGURE 1-1 (cont.)

national technical criteria, and how the risk manager may use or modify these criteria to develop site-specific regulations or permits. The methodology to be discussed in this document falls within the area of Figure 1-1 marked "SLUDGE METHODOLOGY DEVELOPMENT."

1.3. RISK ASSESSMENT IN THE METHODOLOGY DEVELOPMENT PROCESS

As can be seen from Figure 1-1, the methodology development process begins by defining the management practice to be evaluated. Even within a single reuse or disposal option, the manner in which the practice is carried out is highly variable. A definition that encompasses the variety of methods used must be developed. The definition should not be limited to the ideal engineering practice; it should include the types of practices most frequently used. It should not necessarily include practices that are poor or substandard, unless such practices are widespread. The definition of sludge surface disposal, included in Chapter 2 of this document, helps to determine the limits of applicability of the methodology and to identify exposure pathways that may be of concern. However, as shown in Figure 1-1 and as discussed in Section 1.4 below, the methodology described in this document may help to redefine and regulate the practice; as a result, the definition of the practice may be modified.

1.3.1. Hazard Identification and Dose-Response Assessment. Hazard identification requires the evaluation of data that affect whether a chemical poses a specific hazard. It is a qualitative determination, based on information regarding the type of effect produced by exposure, the conditions of exposure, and the metabolic processes within the body that govern chemical effects. Hazard identification includes the determination of whether effects observed under one set of conditions (e.g., laboratory experiments) are likely to occur in other settings (e.g., environmental exposures).

Information on the toxic properties of chemical substances is obtained principally from animal studies and controlled epidemiologic investigations of exposed populations. The use of animal toxicity studies are based on the assumption that effects observed in animals can be extrapolated to effects in humans. Epidemiological studies are also useful in identifying hazards to humans. These studies involve comparing the health status of a group of persons exposed to a causal agent with a comparable unexposed group. In most cases, however, estimates of dose-response relationships are based on animal studies because even good epidemiological studies rarely have reliable information on exposure.

Based on available evidence, the hazard identification requires a decision on whether a chemical should be treated as a carcinogen. Procedures for evaluating the weight of evidence of carcinogenicity have been described in U.S. EPA (1986c). If a chemical is a carcinogen, then a dose-response assessment would consist of the use of Agency-accepted values (U.S. EPA, 1990). If values are not available, the cancer risk estimation procedures published by the Agency may be used (U.S. EPA, 1986c).

If a chemical is not carcinogenic, then the hazard identification and dose-response assessment consist of identifying the critical systemic effect, which is the adverse effect occurring at the lowest dose, and the Reference Dose (RfD), which is "an estimate (with uncertainty spanning perhaps an order of magnitude) of the daily exposure to the human population (including sensitive subgroups) that is likely to be without appreciable risk of deleterious effects during a lifetime" (U.S. EPA, 1990). Methods for deriving RfDs are given in U.S. EPA (1990).

For risks to nonhuman organisms, existing Agency methodologies for assessing possible deleterious effects are used. For aquatic organisms, the methodology uses the existing U.S. EPA water quality criteria for the protection of aquatic life (U.S. EPA, 1984c).

1.3.2. Exposure Assessment. The first step in an exposure assessment is to identify the pathways through which exposure may occur. Exposure pathways are the routes by which chemicals migrate from the reuse/disposal site to the target organ. Individual characteristics of human behavior that affect the likelihood and extent of exposure are given particular scrutiny in the development of models for human exposure pathways. Diversity in individual behavior patterns that affect potential exposure will lead to variation in individual risk. The methodologies described in this document focus on developing estimates of exposure to the "most exposed individual" (MEI)[1], that is, the individual in the exposed population who would experience the greatest health risk. However, a broader spectrum of human behaviors may also be modeled to define the distribution of possible exposure levels and the number of persons exposed to each level. To this end, this report also presents methods for estimating the distribution of exposures within a "most exposed population" (MEP).

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[1] The MEI definition does not include workers exposed through the handling of sludge. It is assumed that workers can take steps to minimize exposure through the use of protective gear. However, agricultural workers may be included in the MEI definition.

Chapter 3 discusses the pathways to be examined and defines the MEI separately for each of these pathways. Specific data used to quantify exposure to the MEI will be given in Chapters 4, 5 and 6, which describe each pathway in detail. For pathways involving risks to nonhuman organisms, the methodology uses the term "most exposed unit" or MEU rather than MEI, but still uses conservative assumptions regarding exposure. This approach should ensure the development of protective criteria.

1.3.3. Risk Characterization. Risk characterization consists of combining exposure assessment and dose-response assessment results. Usually, exposures are estimated by tracing the movement of chemicals from the source to the receptor. In this methodology, deriving the criteria requires the reverse calculation. First, acceptable exposures to the receptor are determined. Next, the concentration in the medium of concern (such as air or water) that would result in acceptable exposures to the receptor are derived. The fate and transport calculations used to estimate chemical concentrations in affected media are then back-calculated in order to estimate the corresponding source concentrations and/or management practices that would result in those media concentrations for the purpose of designing protective regulatory strategies. These calculations are carried out on a chemical-by-chemical basis. Criteria are derived for each chemical assessed and for each pathway. Example calculations for two chemicals, lead and benzene, are provided in this document.

Compiling the data to be used in the methodology is an exercise separate from developing the model. Data on health effects of individual chemicals must be gathered from the scientific literature. In many cases, the U.S. EPA has published values for cancer potency or the Reference Dose. The chemical-specific input parameters for the fate and transport models, such as solubility, Henry's Law constants and bioconcentration factors, must be collected separately from the scientific literature. This information does not appear in the methodology development, except for those chemicals used in the example calculations.

Once data on chemicals of interest have been collected, it is useful to prioritize chemicals for risk characterization. Prioritization may be based on the occurrence of a chemical in sludge, on the likelihood of a chemical to migrate through pathways of concern and on the existence of data gaps that would preclude application of the methodology.

Risk characterization or criteria derivation may be conducted after specific data have been gathered. Criteria may include limits on application rates or concentrations, or restrictions on management practices. When deriving criteria, it is advisable to vary input values over their plausible range to determine the sensitivity of the results to the input values selected.

1.4. POTENTIAL USES OF THE METHODOLOGY IN RISK MANAGEMENT

Risk assessment can be used as input for the risk management process, as shown in part 2 of Figure 1-1. This document does not specify how the risk management should be conducted but does suggest some possible uses of the methodology. A risk manager may evaluate the feasibility of a set of criteria values based on consideration of costs, available technology or other factors unrelated to risk. If certain restrictions on sludge concentrations, as specified by the calculations, would be difficult to achieve, the disposal method could instead be regulated through design standards or required management practices. Following the promulgation of the criteria, it may be possible to evaluate sludge reuse or disposal practices on a site-specific basis, using site-specific data to derive locally applicable standards that would reflect local conditions. Thus, the risk manager may use the methodology as a tool to develop the most reasonable and effective regulatory control strategy for particular sites.

The following chapters discuss one such approach for using the risk assessment methodology. The approach consists of a first step, in which ("Tier 1") numerical criteria are developed for national application, followed by a second step, in which ("Tier 2") site-specific calculations are conducted where appropriate. These criteria are used to determine whether the sludge from each individual facility is of acceptable quality for disposal or long-term storage in an impoundment.

1.5. LIMITATIONS OF THE METHODOLOGY

The limitations of the methods used to derive the criteria are discussed in detail in the text and in tables in the chapters where calculations are presented. Limitations common to all methods are presented here.

Municipal sludges are highly variable mixtures of residuals and by-products of the wastewater treatment process. Chemical interactions could affect the fate, transport and toxicity of individual components; risk from the entire mixture may be greater than the sum of the risks of individual chemicals considered separately. This methodology ignores possible synergistic effects. It should be noted that the U.S. EPA's risk assessment guidelines for mixtures (U.S. EPA, 1986e) caution that a great deal of dose-response information is required before a risk assessment that accounts for chemical interactions may be performed. Future revisions to these documents are likely to include only qualitative discussions of possible toxic interactions.

Transformation of chemicals during sludge disposal (i.e., during combustion or composting) or following release into the environment (i.e., through biodegradation) may result in exposure to chemicals other than those originally found in sludge. In some cases, the methodologies presented in this document may not adequately characterize risks from chemical transformation products.

The methods described in this document do not consider differences attributable to the source of the sludge. For example, the composition of the sludge matrix can be expected to differ between sludge from primary treatment, and sludge from biological treatment processes (e.g., activated sludge systems, trickling filters, and other attached growth systems); these differences are not considered by the methodology. Similarly, potential impacts of lime stabilization on sludge are not discussed in this document, but could be accommodated by the proposed models if sufficient data are available. To the extent that these differences affect the mobility of toxic contaminants in the sludge, the methods proposed here may under- or overpredict actual risks. Similarly, plants receiving a higher contribution of wastewater from industrial sources are treated the same as those that receive no industrial wastewater; the sludge criteria to be derived are independent of the source of the sludge, and refer only to maximum allowable (dry weight) concentrations.

The approach of the methodology described in this report can be used to examine each exposure pathway separately, or to examine simultaneous exposure to more than one pathway. In many cases, it will be unlikely that any single individual would be the most exposed individual through more than one pathway, and adding up the risks across pathways is not recommended. However, if such an individual could receive exposures through more than one pathway, estimates restricted to a single pathway would underestimate risk. Chapters 4, 5, and 6 will outline methods for separate calculation of criteria through individual pathways of exposure. Chapter 7 will provide a simple method for considering these pathways simultaneously.

For nonhuman organisms, the methodology examines risks only to individual specimens and only to aquatic organisms. Population-level or ecosystem-level effects are not examined, because it is assumed that criteria sufficiently protective of individual organisms will also provide sufficient protection at the population or ecosystem level. Although risks to aquatic species are considered where appropriate, risks to non-human terrestrial species have not been examined; it is assumed that criteria sufficiently protective of humans will also protect other terrestrial species for the appropriate pathways of exposure.

2. DEFINITION OF DISPOSAL PRACTICES

2.1. INTRODUCTION

Municipal wastewater treatment works may use one or more levels of treatment (i.e. primary, secondary, or tertiary) to treat wastewater. Each level of treatment provides both greater wastewater clean-up and greater amounts of sludge. Primary treatment processes remove the solids that settle out of wastewater by gravity. These sludges contain 3-7%, 60-80% of which is organic matter. The water content of primary sludge can easily be reduced by thickening or by removing water. Secondary treatment produces a sludge generated by biological treatment processes. These processes remove up to 90% of the organic matter in the wastewater and produce a sludge that typically contains from 0.5-2% solids. These solids are generally more difficult to dewater than primary sludges. The organic content of the solids ranges from 50-60%. Advanced wastewater treatment processes, such as chemical precipitation and filtration, produce an advanced or tertiary sludge. Chemical precipitation uses chemicals to remove organic contaminants and nutrients and to separate the solids from the wastewater. Because these sludges typically contain considerable amounts of added chemicals, the solids content will vary from 0.2-1.5%, while the organic content of the solids will be 35-50%.

All three levels of treatment produce sludge for which reuse or disposal is required. In some circumstances, long-term storage is also desirable. This document discusses a risk assessment methodology for the long-term storage or permanent disposal of sludge in impoundments or lagoons. Included in the methodology are facilities at which sludge accumulates and is stored for long periods as a result of the use of lagoons for wastewater treatment. A lagoon or impoundment is an earth basin used to deposit untreated or digested sludge. Anaerobic and aerobic digestion stabilize organic solids in untreated-sludge lagoons. Stabilized solids settle and accumulate at the bottom of the lagoon (Metcalf & Eddy, Inc., 1979).

No national survey has yet been calculated to identify the number of plants using surface disposal or to estimate the volumes of sludge stored or disposed in this manner. However, 679 plants participating in the 1986 Needs survey reported using forms of sludge treatment and disposal other than landfilling, distribution and marketing, land application, incineration and ocean disposal. Some of these plants may have engaged in practices that fit the definition of surface disposal. Furthermore, 727 plants reported having sludge lagoons, although some of these may have been used for temporary storage or treatment and are thus not included in the surface disposal definition.

The impoundments to be considered in this document can be categorized into three groups: lagoons for long-term storage, lagoons for permanent disposal, and wastewater treatment lagoons in which sludge is stored for long periods. Each of these types of facilities is discussed briefly below.

2.2. LAGOONS FOR LONG-TERM STORAGE OF SLUDGE

Wastewater treatment plants may store sludge from primary, secondary, or tertiary treatment processes in lagoons, with the intention of later exhuming the sludge for ultimate reuse or disposal elsewhere. In most cases, sludge is periodically removed from the facility, although the frequency of such removal will vary among facilities.

Long-term sludge storage may serve a number of purposes. Sludge storage may be an integral part of a plant's overall sludge management plan. For example, sludge may be stored in impoundments over the winter until weather conditions permit land application. Storage of sludge will also tend to decrease water content, thereby decreasing hauling costs when the sludge is exhumed for permanent disposal elsewhere. Smaller plants may store sludge to accumulate quantities that can be practically disposed or reused. Storage impoundments may also be used intermittently, in emergency situations, when normal sludge management operations are overwhelmed.

Although they are not technically "disposal" facilities, lagoons for long-term storage of sludge have been included in methodologies for deriving criteria for "surface disposal" for two reasons. First, this practice may pose health and environmental risks as severe as or worst than those associated with permanent disposal. Second, future plans for removing sludge from "storage" impoundments may be uncertain. As economic or regulatory conditions change, plans for the removal of sludge from "temporary" impoundments have also been known to change. Since the ultimate fate of sludge stored in such impoundments cannot be known with certainty, analysis of risks associated with their continued or permanent existence is appropriate.

2.3. SURFACE IMPOUNDMENTS AS PERMANENT DISPOSAL

In certain cases, sludge may be deposited in on-site impoundments without further planning for removal. Permanent surface disposal may be the only practical disposal option for plants that have accumulated very large quantities of sludge on-site over a number of years. It may also become the de facto disposal method for plants that fail to identify satisfactory final disposal options for the sludge currently stored in lagoons.

2.4. SLUDGE STORAGE AS A COMPONENT OF WASTEWATER TREATMENT

Wastewater treatment lagoons are often used by small wastewater treatment facilities because they are less expensive than alternative treatment processes. Because such lagoons typically receive relatively low volumes of wastewater, they can accumulate sludge in a bottom layer for years (or even decades) before it begins to interfere with the treatment process. Although the main purpose of these lagoons is to treat wastewater before discharge, the lagoons also serve as long-term storage for the sludge, and may fall within the definition of surface disposal of sludge. In addition, the lagoons are sometimes aerated, leading to increased volatilization of sludge contaminants.

2.5. ASSUMPTIONS USED IN RISK ASSESSMENT

The purpose of a sludge storage or disposal impoundment influences the manner in which the sites are managed. For example, sludge in impoundments used for storage between land application seasons will be emptied more frequently than sludge from wastewater treatment lagoons at small plants. All of the surface disposal practices described here pose potential risks from human and non-human exposure. Each practice also possesses unique features that affect its associated risks. Long-term storage impoundments undergo periodic sludge removal, which may increase or decrease movement of sludge contaminants into the environment. Permanent disposal impoundments might present risks through future land uses of the disposal site (although these risks are not considered by the present methodology). There follows a brief listing of assumptions used for estimating risks from these sludge management practices:

1. Sludge storage or disposal occurs on plant property, and the public has no access to the affected areas.
2. Sludge impoundments will be required to have berms, dikes or other surface runoff controls that effectively eliminate significant risks of exposure from flooding or accidental releases.

3. Periodic removal of sludge from long-term storage lagoons allows their continuous use over extended periods of time.

4. Sludge that accumulates in long-term storage or wastewater treatment lagoons is removed prior to plant closure, and before any conversion of the land to other uses can take place.

5. Sludge deposited in permanent disposal lagoons is never exhumed.

2.6. CONDITIONS AFFECTING RISK

A number of environmental conditions and management practices affect the risks posed by long-term sludge storage impoundments. Management or design requirements can be developed, based on an evaluation of the following factors, to minimize risks from surface disposal:

1. Concentrations of contaminants in the sludge will influence the magnitude of risk posed by long-term sludge storage or permanent disposal.

2. Physical characteristics, such as distance to groundwater and surface water, soil type and other geohydrologic features of the site, and proximity to human or non-human populations will influence the rate of migration of contaminants and the potential for exposure.

3. The length of time sludge is stored (i.e., the frequency of sludge removal from the impoundments) may affect the mobility of sludge contaminants. Frequent removal may decrease the length of time sludge contaminants are available for leaching, but could also disturb the underlying stabilized sludge layer that may inhibit contaminant movement from unlined impoundments.

4. The use of synthetic or clay liners will reduce the potential for contaminant movement into groundwater.

5. The use of berms and proper siting can reduce risks associated with surface runoff and flooding.

6. For permanent disposal facilities, restrictions on future access and land use through a notice in the deed may be required once the facility is closed in order to prevent inadvertent direct human contact with disposed sludge. Alternatively, the sludge disposal area may be encapsulated at closure in order to prevent future direct contact with sludge if land is converted to another use.

7. Aeration of wastewater treatment lagoons will affect the volatilization and biodegradation rates of the organic sludge contaminants.

Improper management practices at a particular facility can result in additional risks to human health or the environment; the methodology presented here does not include techniques for assessing risks associated with such practices. Differences in potential risk associated with siting of surface disposal facilities are considered by the methodologies insofar as they can be represented by the input parameters required for model calculations.

2.7. SUMMARY

The methodology presented in the following chapters is designed to derive criteria for lagoons used for both long-term storage and permanent disposal of sludge. The methods are intended to apply to all types of municipal wastewater sludges, regardless of the extent of industrial contribution to the treated wastewater, and regardless of the type of wastewater treatment processes that generate the sludge. Risks from these surface disposal facilities can occur through a variety of pathways of potential human and non-human exposure. Chapter 3 of this document will discuss the selection of pathways for inclusion in this methodology, and the identification of "most exposed individuals" and "most exposed populations" at greatest risk through each pathway. Chapters 4, 5, and 6 will provide detailed discussion of methods for assessing potential risks and deriving criteria for the three exposure pathways thought to be of most concern: groundwater, surface water, and air.

3. EXPOSURE PATHWAYS AND MOST EXPOSED INDIVIDUALS (MEIs)

3.1. INTRODUCTION

Human or environmental exposure to contaminants from wastewater sludge can occur through a number of pathways. The pathways of greatest concern for surface sludge disposal will depend on the purpose of surface disposal (e.g., storage vs. wastewater treatment) and the management practices employed. This chapter describes pathways of potential concern for each of the three types of surface disposal, using the definitions developed in Chapter 2, and identifies management or design practices that may influence exposure through these pathways. When a particular exposure pathway is identified as a pathway of concern for a given management practice, the assessment methods described later in this report should be used to determine whether criteria are needed. Identification of pathways of concern for a given management practice should not be interpreted to mean that criteria will necessarily be required for that practice.

The methodology described in this document considers three types of pathways of potential exposure: a pathway involving the transport of sludge constituents through groundwater, pathways involving subsequent transport through surface water, and a pathway involving the volatilization of organic constituents to ambient air. Risks associated with each of these pathways will be examined later in this report. For each pathway identified, risk to the most exposed individual (MEI) will be evaluated. The MEI is defined as the individual in the general population who would experience the greatest health risk, either because of the expected magnitude of the individual's exposure to contaminants or because of physiological or behavioral characteristics that make the individual particularly susceptible or sensitive to contaminant exposure. For certain pathways, aquatic species may be the organisms at greatest risk from exposure to sludge contaminants; for these pathways, the most sensitive aquatic species will be considered the MEI or most exposed unit (MEU) of interest. Although the MEI (or MEU) is a hypothetical individual, care should be taken to make the definition realistic. For each disposal option and exposure pathway, formulas are presented to calculate regulatory criteria that adequately protect the appropriate MEI or MEU. It is reasoned that as long as the risk assessment procedures can reasonably estimate the risks to these individuals, then the quantification of lesser risks experienced by other individuals is not required.

Two problems may arise from reliance on such an approach. First, physical identification of the actual MEI is rarely feasible; the MEI to be considered when setting criteria is therefore a theoretical construct. Where large modeling uncertainties exist, the compounding of conservative assumptions in the MEI definition or exposure models can produce extreme MEI scenarios that result in unnecessarily strict criteria. Conversely, seemingly reasonable "worst case" scenarios may be found, upon closer inspection, to predict lower levels of exposure than those encountered by actual individuals in the exposed population, especially when the size of the exposed population is large and potential variability in individual behavior is great. Choices for defining the "most exposed individual" are directly linked with the ultimate stringency of regulatory criteria; inappropriate choices are likely to lead to inappropriate criteria.

A second problem with using an MEI approach to set regulatory criteria is the difficulty of maintaining comparability in MEI definitions across all sludge management practices and exposure pathways. If compliance with criteria for one sludge management practice is more difficult than compliance with criteria for another, sludge managers will be encouraged to use the practice with the more lenient criteria. Changes in management practice will serve the public health and environmental quality only if the more strictly regulated option for sludge reuse or disposal is indeed riskier than the less strictly regulated one. Over-regulation of one practice coupled with under-regulation of another practice may actually increase health or environmental risks if sludge managers shift from alternatives with lower risks to those with higher risks. Such disparities, and their consequences, are most likely if MEI definitions for different sludge reuse or disposal practices are not comparable.

Avoidance of such unwanted consequences requires a systematic approach to defining the exposure scenarios and the most exposed individuals for which sufficiently protective criteria are to be derived. The approach should have two goals: (1) to derive realistic MEI definitions that are both realistic and sufficiently protective, and (2) to achieve comparability across MEI definitions applicable to different sludge management practices. One such approach involves consideration of "most exposed populations" (MEP) for each sludge management practice and for each potential pathway of exposure. The approach involves estimating the size of the populations exposed to various levels of sludge contaminants through each exposure pathway. Based on estimates of national distributions of various relevant parameters, criteria can be derived that are sufficiently protective of populations of specified (non-zero) estimated sizes. The natural extension of these techniques is to use these distributions to derive an MEI definition appropriate for a single (theoretical) individual with the highest true potential for exposure. Application of these methods to the derivation of criteria for surface disposal facilities will be discussed in more detail in Chapters 4, 5 and 6.

The precise definition of the MEI will depend on the assumptions and requirements for each management practice described in Chapter 2. This chapter will qualitatively define the MEI for each potential pathway of concern. The MEI will be described quantitatively, using reasonable worst-case exposure assumptions, in the following chapters where criteria calculation methods are given. As discussed in Chapter 1, occupational exposures will not be considered in this analysis. Neither will the analysis consider hazards that are not related to routine migration of sludge contaminants from the impoundment into the environment (e.g., exposure to pathogens, potential surface water contamination by flooding, risks of releases caused by earthquakes, underground gas migration and risks of explosion, nuisance concerns, and general environmental concerns like global warming, ozone depletion, or vegetative distress).

Possible pathways through which humans or other organisms may be exposed to contaminants from surface-disposed sludge include:

1. sludge-groundwater-wells-drinking water;
2. sludge-groundwater-surface water-drinking water;
3. sludge-groundwater-surface water-consumption of fish;
4. sludge-groundwater-surface water-exposure of aquatic life;
5. sludge-volatilization-downwind transport-inhalation.
6. sludge-surface runoff-surface water-drinking water;
7. sludge-surface runoff-surface water-consumption of fish;
8. sludge-surface runoff-surface water-exposure of aquatic life; and
9. sludge-wind transport of particulates or aerosols-inhalation
10. sludge-soil (site conversion) - humans

The methodology presented in this document will be restricted to consideration of those pathways associated with the transport of contaminants through groundwater (and subsequently through surface water) or with the volatilization of organic contaminants. Potential exposure associated with surface runoff from the facilities, from possible wind transport of particulates or aerosols, and from pathways associated with future conversion of the site to residential uses will not be included.

3.2. GROUNDWATER PATHWAY

Contaminants in sludge currently stored or disposed in impoundments can leach into the groundwater underlying the site. Sludge contaminants may subsequently migrate into wells from which drinking water is obtained. Although all three types of surface disposal units under consideration potentially pose risks through the groundwater pathway, the rate and extent of contaminant migration from sludge impoundments may be greater than from other kinds of sludge disposal sites because of the presence of liquid in the impoundment. The presence of synthetic liners may retard or delay migration of contaminants. The MEI for this pathway would be an individual who obtains all drinking water over the course of a lifetime from a well adjacent to the wastewater treatment plant boundary.

If contamination of groundwater resources is sufficiently widespread, a number of wells may be affected by sludge contaminants. If more than one well is potentially affected, and the level of contamination differs significantly among the different wells, then a more detailed analysis of the extent to which exposure varies among individuals in the exposed population (an analysis of the MEP) can provide additional information of potential usefulness to the regulatory process.

3.3. SURFACE WATER PATHWAY

Sludge contaminants may enter surface water through direct surface runoff from sludge impoundments or waste piles. However, it is assumed that since the purpose of the impoundments is sludge containment, berms and other surface runoff controls will be required at all surface disposal sites. However, sludge contaminants may enter groundwater and subsequently discharge into surface water. Contaminants in surface water can accumulate in the body tissues of aquatic organisms. Endpoints of concern include the direct exposure of aquatic life to contaminated water, and human ingestion of contaminated surface water and fish. The ecological endpoint of greatest concern would be the aquatic species most sensitive to the presence of the contaminant in the water. The human MEI would be defined as that individual who obtains all drinking water from an intake downstream from the point of contaminant discharge into the surface water body, or who consumes fish caught near the point of contaminant discharge into the water body. Criteria sufficiently protective of aquatic species and of the human MEI are assumed to be sufficient for protection of nonhuman terrestrial species.

If more than one human is exposed, then levels of potential exposure are likely to vary within the exposed population. In this case, a more detailed analysis of the MEP can provide guidance in selecting an appropriate definition for the human MEI, and can provide other information useful to the regulatory process.

The groundwater-surface water-drinking water, groundwater-surface water-fish consumption, and groundwater-surface water-exposure of aquatic life pathways may be important pathways of exposure for any of the surface disposal practices under consideration. These pathways may be particularly important whenever surface disposal is practiced in close proximity to surface water resources. When using the methodology for the purpose of setting national criteria, it should be assumed that surface waters near surface disposal sites may be used for both drinking and fishing.

3.4. AIR PATHWAY

Volatile sludge contaminants may be emitted into the air and may be dispersed downwind, where they may be inhaled by receptors. The MEI is defined as the downwind receptor closest to the sludge surface disposal area. The distribution of potential exposure over the entire exposed population may also be of interest. To obtain an estimate of the distribution of exposure, air modeling can be used to derive estimates of air concentrations surrounding a facility as a function of distance; this information is coupled with population data to derive a distribution of air concentrations as a function of the number of persons exposed.

While volatilization may occur from any of the sludge surface disposal sites described in Chapter 2, this pathway may be especially important for aerated wastewater treatment impoundments. Aeration will increase the rate of volatilization, resulting in higher downwind concentrations.

4. DERIVATION OF CRITERIA FOR THE GROUNDWATER PATHWAY

This chapter describes a methodology for deriving criteria, based on human exposure through the groundwater pathway, for the storage or disposal of sludge in surface impoundments. The approach is based on methodologies for assessing risks to human health, and can be used for both national and site-specific applications. A risk assessment methodology for groundwater contamination resulting from the landfilling of municipal sludge (Development of Risk Assessment Methodology for Municipal Sludge Landfilling, U.S. EPA, 1986a) has also been developed as a part of this document series; the present methodology differs from that earlier effort in two important respects. Surface disposal facilities differ from landfills in that the sludge deposited in an active lagoon is likely to contain more liquid than does sludge deposited in a landfill. Consequently, the "source term" for estimating groundwater contamination beneath a surface disposal facility will differ from the source term for a landfill. In addition, the methodology proposed in this document departs from that proposed for landfills because it selects a different mathematical model for estimating flow and contaminant transport through the unsaturated soil zone. Nevertheless, some elements of the two risk assessment methodologies are identical; to avoid lengthy repetition of text, this document occasionally refers the reader to the landfill methodology document for additional discussion of details shared by the two methodologies.

4.1. OVERVIEW OF THE METHOD

Water leaking from the bottom of surface disposal sites can contain dissolved contaminants from the sludge. These contaminants can be carried downward by water percolating through the unsaturated zone beneath the facility, until they reach an underlying aquifer. Within the aquifer, they can be transported beyond the property boundaries of the facility where the contaminated water may be withdrawn through private or public wells for human consumption. As with other pathways of potential human and wildlife exposure to be discussed in this document, the methodology for deriving criteria based on the groundwater pathway involves a two-tiered approach. Tier 1 derives numerical sludge criteria for national application, whereas Tier 2 provides criteria tailored to conditions at specific surface disposal sites.

Tier 1 analysis begins with the calculation of reference water concentrations (RWC) for each sludge contaminant of concern; these are calculated using methods to be described in Section 4.5. Next, the analysis uses a mathematical model of a "generic" surface impoundment scenario to estimate the maximum concentration of each sludge contaminant that can be allowed in the impoundment if contaminant concentrations in groundwater beneath or near the facility are not to exceed the RWC. These maximum allowable concentrations are used as criteria for regulating sludge quality for all surface disposal facilities. The scenario is selected to represent reasonable worst case conditions, so that the associated risks are as high as or higher than those to be expected at actual facilities. If desired, these maximum allowable concentrations, or sludge criteria, can be estimated separately for different aquifer classes.

If sludge produced by a particular publicly owned treatment works (POTW) fails the numerical criteria derived by Tier 1 analysis, then a site-specific (Tier 2) analysis is required. Tier 2 analysis begins with the reference water concentrations derived in the first step of Tier 1. Using the same mathematical model employed in Tier 1, but with site-specific input parameter values, the analysis computes maximum sludge concentrations that can be allowed at the particular facility if reference water concentrations are not to be exceeded. As with Tier 1, separate requirements can be imposed according to the class of aquifer involved.

Both methodologies are based on a mathematical model that describes the migration of sludge contaminants from an impoundment to an underlying aquifer, lateral movement of contaminants through the aquifer to a nearby well, and withdrawal of drinking water from the well for human consumption. The property boundary may be selected as the point of compliance, since drinking water wells could be constructed at that location and outward, and contamination of the wells could result in potential impacts to public health. This selection would be based on the premise that any potable water supply beyond the property boundary must be maintained at healthful levels for potential future uses even if there are no current wells immediately off site. Alternatively, groundwater directly beneath the site might be selected as the point of compliance if the site is located over a Class I aquifer.

The mathematical model includes five analytical steps:

1. estimation of the quantity of water seeping from the bottom of the impoundment and of contaminant concentrations in that water,
2. estimation of the extent to which contaminant concentrations are reduced during transport through the unsaturated zone,
3. estimation of the extent to which concentrations change during horizontal contaminant transport to a receptor well,
4. estimation of potential human exposure based on well-water concentrations, and
5. evaluation of potential human health risk, based on comparisons of expected exposure to risk reference doses for non-carcinogenic contaminants, and on the use of potency values for carcinogens.

Details for each step in the methodology are provided in Sections 4.3 and 4.4.

Numerical criteria are specified as limits to the dry-weight concentrations of contaminants in sludge. These sludge criteria must be set so that any environmental contamination resulting from surface disposal of sludge would not exceed health-based reference concentrations. To derive criteria for the storage or disposal of sludge at a particular site, the five steps of the methodology must be linked in reverse order: based on reference water concentrations in a drinking water well, the methodology is used to estimate maximum allowable concentrations in sludge as it enters the impoundment. Facilities managing sludge must compare the sludge concentration of each contaminant with the Tier 1 criteria. If all contaminant concentrations pass the criteria, then the facility's application is accepted. If any one contaminant exceeds the Tier 1 criteria, then a Tier 2 analysis can be performed using site-specific data to derive (presumably less conservative) sludge criteria more applicable to conditions at the particular site.

4.2. ASSUMPTIONS

Application of the methodology presented in this document requires several simplifying assumptions; these are summarized in this section and in Table 4-1, but will be discussed in greater detail in Section 4.3. As can be seen from Table 4-1, calculations are based on a source term of seepage volume and contaminant mass flux into the unsaturated and saturated soil zones beneath a lagoon. Representation of this source term depends on the certainty with which the future operation of the lagoon can be predicted; for storage facilities from which sludge is occasionally removed,

TABLE 4-1

Assumptions for Methodology to Analyze the Groundwater Pathway

Functional Area	Assumption	Ramifications
Source Term	At sludge storage facilities for which the active lifetime cannot be predicted with certainty, contaminant flux through seepage can be described as steady-state, with a renewable supply of contaminant mass.	May overpredict loading to aquifer after facility ceases to accept sludge.
	For permanent disposal facilities at which the timing of the final sludge deposit is known, contaminant flux through seepage can be described as a square wave of duration determined by the mass of contaminant contained within the impoundment.	Overpredicts duration of high concentration period, but may underpredict peak concentration if mass is conserved. Neglects presence of low concentration tail.
	Ratio of solids to liquids at bottom of impoundment is 10:1. Dry-weight concentration of contaminant in sludge of bottom layer equals that of sludge entering the facility. Dissolved concentration can be predicted from an equilibrium distribution coefficient.	Unknown.
Unsaturated Zone Flow	One-dimensional flow in the vertical direction.	Overpredicts concentration since it ignores horizontal dispersion.
	Water flow is steady state; upper boundary has constant flux of contaminant mass.	May overpredict loading after impoundment ceases to accept sludge.
	Soil characteristics are constant with depth for any layer.	Unknown.
Unsaturated Zone Transport	Attenuation of organics is related to soil organic fraction only.	Overpredicts contaminant velocity for soils with low organic content where mineral interaction may predominate.
	All adsorption is reversible.	Overpredicts concentration arriving at aquifer.
	Degradation of organic contaminants is first order.	Unknown.

TABLE 4-1 (cont.)

Functional Area	Assumption	Ramifications
Saturated Zone Transport	Potential effects of "mounding" can be approximated with a superimposed velocity term, estimated from quantity of seepage, perimeter of site, and depth of aquifer.	Likely to overestimate flow velocity in aquifer. Likely to overestimate velocity and concentrations at the receptor well.
	Geochemical reactions and other "pseudo-decay" processes are ignored for metals.	Overpredicts concentrations of metals.
	Degradation of organic contaminants is first order.	Unknown.
	Receptor well is located directly down-gradient of the site, at a distance defined by the property boundary.	Likely to overpredict concentrations in actual wells.
Transport to Surface Water	Direction of groundwater flow is toward surface water, perpendicular to the bank of surface water body.	Will overpredict loadings where actual direction is different.
	Average distance of travel is approximated by shortest distance from site to surface water.	Likely to overpredict loadings.
	Total mass of contaminant in seepage from the impoundment eventually reaches the surface water body, except that lost to degradation.	May overpredict loading to surface water body.
	Vertical and transverse dispersion of contaminant (parallel to stream bank) can be ignored when calculating average loading.	Should not affect results if loading along entire stream bank is to be considered, and if the entire contaminant plume is intercepted by the stream.

operation can continue indefinitely, and the source of seepage and contaminant mass can be renewable. For permanent disposal facilities, the moisture and contaminant mass supplied by the sludge are considered finite. Maximum loading to groundwater is assumed to occur once a sludge layer has accumulated on the floor of the impoundment, and solids concentrations are highest. Dissolved concentrations of contaminant within this sludge layer, and within seepage from the floor of the lagoon, are assumed to be predictable based on equilibrium distribution coefficients and a conservative 10:1 ratio of solids to liquids within the sludge layer. As proposed, the methodology does not consider potential losses of contaminant mass to biodegradation or volatilization during containment within the impoundment. Although methodologies proposed in Chapters 4 and 6 could be linked to provide a mass balance between these competing loss processes, the methodologies proposed in Chapter 6 for estimating volatilization and biodegradation are not intended to provide an accurate depiction of conditions within the sludge layer on the floor of an impoundment.

As will be explained in further detail in Section 4.3, flow and contaminant transport through the unsaturated zone are represented by a one-dimensional model that allows consideration of multiple soil layers, each with soil characteristics that do not vary with depth. With the unsaturated zone, the attenuation of organic contaminants is predicted based on longitudinal dispersion, an estimated retardation coefficient derived from an equilibrium partition coefficient, and a first-order rate of contaminant degradation. Potential effects from local elevation of the water table are considered by the model when estimating contaminant attenuation during transport within the unsaturated zone. Concentrations of metals are assumed to be conserved during transport through the unsaturated zone.

Contaminant migration through the saturated soil zone is estimated with a model that includes adjustments for likely effects of "mounding" associated with significant seepage beneath some facilities. Consideration of these effects within a one-dimensional model of contaminant transport through the saturated zone requires some simplifying assumptions about the velocity of contaminant transport and the extent of contaminant dilution by the aquifer. As in calculations for the unsaturated zone, degradation of organic contaminants are assumed to be first order during transport through the aquifer. Geochemical reactions are ignored for metals, leading to the likely overestimation of expected concentrations of metals in groundwater at a receptor well location. The receptor well is conservatively assumed to be located in the down-gradient direction at a minimum distance defined by the property boundary.

Estimation of expected loadings of contaminant to nearby surface water bodies is based on the assumptions summarized above, as well as a few additional assumptions particular to this pathway of potential exposure. The direction of groundwater flow is assumed to be toward the surface water; the time of travel and extent of contaminant dispersion during transport to the surface water are determined by the aquifer medium and by the shortest distance from the site to the lake or stream. Contaminant loading to the surface water body is predicted without regard to dispersion of contaminant in directions parallel to the shoreline of the surface water body. Further details for the assumptions required for the methodology will be provided in Sections 4.3 and in Chapter 5.

4.3. CALCULATIONS

This section provides a description of the calculations and model used for deriving criteria. The methodology is described within the context of a site-specific application for a particular surface disposal site. As mentioned above, however, the same methods can also be used for Tier 1 calculations to derive national, numerical criteria.

4.3.1. Source Term. Each contaminant transport pathway begins with a source term and ends with a receptor or point of exposure. In evaluating sludge disposal alternatives, the sludge itself is the source term through which the various contaminants are introduced into the environment. To quantify risks attendant to the storage or disposal of sludge in surface disposal facilities, it is necessary to determine the mass of contaminant released as a function of time.

To characterize the behavior of sludge in an impoundment, it is useful to conceptualize the lifetime of a surface disposal facility in two phases. In the first (or "active") phase, sludge is continuously or periodically added to the facility. For the special case of a wastewater treatment lagoon, the sludge will accumulate as a result of the settling of solids from wastewater. For other types of impoundments discussed in Section 2.1, sludge solids simply accumulate as a result of repeated sludge deposits. During the active phase of the impoundment, the reservoir of contaminant mass contained in the impoundment can increase as a result of sludge accumulation. An accumulating sludge layer on the bottom of the impoundment might result in increased concentrations of dissolved sludge contaminants localized at the bottom of the impoundment. On the other hand, the presence of an anaerobic sludge layer at the bottom of a lagoon may reduce contaminant concentrations in leachate and may slow the rate of seepage from an impoundment (Kehew et al., 1983; U.S. Geological Survey, 1988b). Average rates of seepage from a particular lagoon can be estimated from a mass balance that includes inflow, outflow, precipitation and evaporation. Typical rates of seepage beneath municipal lagoons are listed in Table 4-2, taken from U.S. EPA (1987b).

In the second, or "inactive," phase of the facility's lifetime, new additions of sludge are no longer received by the impoundment. For those impoundments used only for storage of sludge (e.g., wastewater treatment lagoons that accumulate sludge, or impoundments used for temporary holding and dewatering), accumulated sludge is periodically removed, and the facility may never reach the inactive phase without being emptied. Impoundments used for permanent sludge disposal, however, are eventually closed without removal of their contents. Contaminant mass within the impoundment may decrease as a result of volatilization, biodegradation, seepage to groundwater or other loss processes. If water seepage beneath the facility exceeds local precipitation less evaporation, the contents of the impoundment will lose moisture until the rate of seepage reaches equilibrium with the rate of recharge. As the moisture content of the impoundment decreases, the reduced rate of seepage beneath the facility might result in a reduced flux of contaminants to groundwater, but that reduction could be offset at least partially by increased concentrations of dissolved contaminants that might leach from sludge with a higher solids content.

Because of uncertainties about the behavior of the bottom layer of sludge in impoundments, and because future plans for sludge removal or addition cannot be known with certainty for many surface impoundments, separate modeling of the expected active and inactive phases of a particular impoundment is in many cases infeasible. For these reasons, the methodology presented here combines estimates of steady-state seepage from an active impoundment with conservative estimates of leachate concentrations that may be more typical of inactive lagoons.

Calculations for both Tier 1 and Tier 2 begin with three conservative assumptions for estimating expected concentrations of contaminant in seepage from a lagoon. First, they assume that

TABLE 4-2

Summary of Measured Seepage Rates From Municipal Lagoon Systems[a]

Water depth (feet)	Lagoon Type	Underlying Soil	Seepage Rate	
			in/day	l/m^2-hour
5	Facultative	Heavy silty clay	0.3	0.32
6	Facultative	Light silty clay	0.29	0.31
5	Facultative	Alkaline silt	0.65	0.69
6	Facultative	Fine sand	1.2	1.3
6	Facultative	Gravel and silt	1.3	1.4
-	--	Sandy soil	0.35	0.37
-	--	Sand and gravel	0.61[b]	0.65
-	--	Sandy soil	0.34	0.36
-	--	Clay loam and shale	0.3	0.32
-	Maturation	Mica and schist	0.06-0.23	0.06-0.24
5	Facultative	Silt, sand, marl	0.18	0.19
5	Facultative[c]	Sand, silt, marl	1.07	1.13
5	Evaporation[d]	Sand, silt, marl	0.04-0.11	0.04-0.12
-	Facultative	Sandy soil	0.12	0.13

[a] Source: U.S. EPA, 1987b
[b] Includes net precipitation/evaporation
[c] Used intermittently
[d] Sealed with bentonite and soda ash

sludge at the bottom of an active or inactive impoundment contains a low water content. Values for the water content of drained sludges are presented in Table 4-3; most sludges described by the table retain water at a ratio of 12:1 to 24:1. The use of a 10:1 ratio might therefore represent a reasonable "worst case" value appropriate for the bottom layer of an inactive impoundment. Second, the calculations assume that the contaminant mass in the sludge and water mixture on the lagoon floor is partitioned at equilibrium between dissolved and adsorbed phases. It is assumed that this partitioning is described by an appropriate partition coefficient (k_d) for each contaminant of concern. Then the dissolved concentration of contaminant can be related to the total (dry weight) sludge concentration by:

$$C_l/N = 1/(k_d + r_{sl}^{-1}) \qquad (4\text{-}1)$$

where:

C_l = concentration of the contaminant in liquid phase at the bottom of the impoundment (mg/l or g/m^3)

N = dry weight concentration of contaminant i in sludge at the bottom of the impoundment (mg/kg)

k_d = equilibrium partition coefficient for the contaminant (l/kg)

r_{sl} = ratio of solids to liquid in sludge at the bottom of the impoundment, conservatively assumed to be 0.1 (kg/l)

Values for the partition coefficient (k_d) for each contaminant of concern can be derived from the literature, obtained from Appendix C of U.S. EPA 1986a, calculated with the CHEMEST procedure from the Graphical Exposure Modeling System (GEMS) maintained by the U.S. EPA Office of Pesticides and Toxic Substances (U.S. EPA 1988c), or can be estimated empirically for sludge using methods described in DeWald and Phillips (1989).

An alternative method for predicting leachate concentrations would be to use actual measurements of contaminant concentrations in water drained from each individual facility's sludge. This method avoids the necessity of relying on theoretical estimates of leachate concentrations, but results are likely to depend on the initial solids content of the sludge received by the impoundment, and might not reflect conditions at the bottom of a lagoon that has accumulated a sludge layer.

TABLE 4-3

Water Content of Sludges from Various Treatment Processes*

Sludge Type	Water Content (%)	Water : Solids
Primary sludge	95	19
Digested primary sludge	94	16
Trickling filter	92	12
Chemical precipitation	92	12
Primary and activated sludge	96	24
Digested primary and activated sludge	94	16
Activated sludge	98.5	66
Septic tank-digested activated sludge	90	9
Imhoff tank-digested activated sludge	85	6

*Source: U.S. EPA, 1978

Characterization of the source term also requires estimating the time over which the contaminant will be present in the leachate. Sludge contains a finite mass of contaminant that can be mobilized in leachate. For some contaminants, that mass is less than the total mass in the sludge because of irreversible adsorption or other binding mechanisms.

As long as an impoundment is active, the mass of sludge contaminant it contains can be periodically restored, and will not be permanently depleted. For an inactive permanent disposal impoundment, however, the mass of available sludge contaminant will be depleted over time. As discussed above, this methodology conservatively assumes that the end of the active phase for most impoundments cannot be predicted with certainty and, therefore, assumes that an impoundment continues operation indefinitely. This assumption is represented by steady-state loadings of contaminant to the unsaturated zone beneath an impoundment.

For cases where a reasonable determination of the duration of an impoundment's active lifetime can be accomplished, however, the model will allow the applicant to model loadings to groundwater as a pulse, or series of pulses, of finite duration. For such a facility, calculation of pulse time requires determination of total contaminant levels in the contents of the inactive facility, rates of decay for contaminant concentrations, and the time path of seepage volume as seepage rates decrease with decreasing moisture content in the impoundment. Derivation of pulse times is based on mass conservation considerations. Details of the necessary calculations are provided in U.S. EPA (1986a).

4.3.2. Unsaturated Zone Flow and Transport. No one method or model for estimating water flow and contaminant transport through the unsaturated zone is appropriate for application to all cases. In general, criteria used to select models for this methodology are as follows:

1. the method should be simple and amenable to widely available equipment;
2. the data required by the method should be generally available; and
3. the method should be applicable to a wide range of problems or sites.

The first criterion is important if the model is to be used for numerous site-specific applications in a variety of locations. As model complexity increases, demands on the skill and judgement of the modeler also increase, as does the difficulty of applying the model consistently and equitably from location to location. In addition, large numerical model codes can require expensive or specialized computing equipment that may be unavailable in some locations. The second criterion, that the data required by the method should be generally available or that the data can be estimated, is important if expensive, specialized site studies are not to be required in support of each application. For each site, required data should be already available, should be easily obtainable or measurable, or should be amenable to reasonable approximation based on literature sources. The third criterion is that the method be generally applicable to a wide range of problems or sites. Potential sites exist across the United States and, therefore, may be characterized by a range of values. It is impossible to select any single method that is optimal for all sites, especially given the constraints of the first two criteria. Methods should be selected that can be used for the wide range of values potentially encountered, without requiring different approaches for each setting.

Simulation of contaminant transport through the unsaturated zone can be accomplished by models with varying degrees of sophistication. Complex three-dimensional models provide the best available simulation of contaminant transport beneath a surface impoundment, but these models require more detailed geohydrological data than are typically available at existing sludge management facilities. In addition, they require considerable modeling expertise for proper execution and interpretation. A one-dimensional vertical unsaturated zone model is simpler to use, requires fewer input data and is easily applied to a wide variety of sites. The methodology proposed herein therefore relies on a one-dimensional model for simulating contaminant movement through the unsaturated zone.

The present methodology links two well-accepted models for unsaturated and saturated zone flow and transport. The Vadose Zone Flow and Transport finite element module from the RUSTIC model (U.S. EPA, 1989a,b) is used to estimate flow and transport through the unsaturated zone, and the AT123D analytical model (Yeh, 1981) is used to estimate contaminant transport through the saturated zone. The combination of these two codes creates a simple-to-use model appropriate for deriving sludge criteria.

Three candidate models for estimating flow and transport through the unsaturated zone were considered for use in the present methodology: the CHAIN model (van Genuchten, 1985) as used in

U.S. EPA (1986a), the LEACHM model (Hutson and Wagenet, 1985), and the VADOFT module from RUSTIC. CHAIN was rejected because of previous problems encountered when using the model to derive criteria for sludge monofills. Review of supporting documentation suggests that both LEACHM and VADOFT are well suited for adaptation to model the unsaturated zone beneath a surface impoundment; VADOFT was selected in part because it is supported by the U.S. EPA Office of Research and Development in Atlanta. U.S. EPA (1989a) reports that tests on benchmark problems show that results from VADOFT are in good agreement with results from the more sophisticated two dimensional finite element codes UNSAT2 (Davis and Neuman, 1983) and SATURN (Huyakorn et al., 1984). The Pesticide Root Zone Model (PRZM) is used by the RUSTIC model to simulate the movement of pesticides through the soil surface, and runoff to surface water. It is not appropriate for use in modeling contaminant migration beneath a surface impoundment. The remaining component of RUSTIC, or SAFTMOD, simulates the movement of contaminant through the saturated zone, and could have been appropriate for this methodology, but was rejected in favor of AT123D, for consistency with other methodologies described in this series.

The ultimate goal of this methodology is to determine the maximum contaminant concentration in sludge that will result in contaminant concentrations in groundwater that do not exceed reference water criteria. As mentioned earlier, the proposed model executes in two steps; results from the unsaturated zone flow and transport module are passed as input to the saturated zone module. The input requirements for the unsaturated zone module include various site-specific and geologic parameters and the leakage rate from the bottom of the surface disposal facility. It is assumed that the concentrations of various contaminants entering the unsaturated zone beneath a facility can be represented by the concentrations derived with methods discussed in Section 4.3.1. Results from the unsaturated zone analysis give the flow velocity and concentration profiles for each contaminant of interest. These velocities and concentrations are evaluated at the water table, converted to a mass flux and used as input to the saturated zone module.

A complication in estimating contaminant movement beneath a surface impoundment arises from the inherent multi-dimensionality of water flow and contaminant transport. Seepage from the surface impoundment can cause local elevation of the water table if rates of seepage from the lagoon exceed natural rates of aquifer recharge in the surrounding area. Such elevation of the water table, or mounding, has two implications for the expected concentrations of sludge contaminants at a receptor well. First, the reduced vertical distance between the impoundment and the local water table will result in decreased time of travel for water moving between the impoundment and the saturated zone, and changes the extent of soil saturation between the impoundment and the floor of the aquifer. The second effect of mounding in the water table is an increased hydraulic gradient in the aquifer between the disposal site and the down-gradient receptor well. This change in the gradient will increase the expected rate of horizontal transport of the contaminant through the saturated zone.

To accommodate these two effects in the model calculations, this methodology adapts the approach used in the RUSTIC model. The first component of the model performs calculations for a vertical column containing both unsaturated and saturated zones, and predicts the extent to which the elevation of the water table will be increased by the flux of water from the impoundment. Once the vertical column problem has been solved for mass and water fluxes at the water table elevation, the second model component (AT123D) simulates the movement of contaminants through the saturated zone, with adjustments to represent increased elevation of the water table. Unlike RUSTIC, however, the present methodology does not allow for partial feedback between the unsaturated and saturated zone components of the model; the saturated zone is represented separately by an analytical transport model.

The flow system in the vertical column is solved with the VADOFT component of the RUSTIC model, which is based on an overlapping representation of the unsaturated and saturated zones. The user must specify a water flux at the soil/liquid interface at the bottom of the impoundment, which defines the top of the unsaturated zone in the model. In addition, a constant pressure-head boundary condition must be specified for the bottom of the unsaturated zone beneath the lagoon. This pressure-head is chosen to be consistent with the expected pressure head at the bottom of the saturated zone, without consideration of the added flux leaking from the impoundment. Transport in the unsaturated zone is then determined using the Darcy velocity (V_d) and saturation profiles from the flow simulation. From these, the transport velocity profile can be determined.

Although limited to one-dimensional flow and transport, the use of a rigorous finite-element model in the unsaturated zone allows consideration of changes in water-table level and also allows consideration of depth-variant physical and chemical processes that would influence the mass flux entering the saturated zone. Among the more important of these processes are advection (which is a function of the Darcy velocity, saturation and porosity), mass dispersion, adsorption of the leachate onto the solid phase, and both chemically and biologically induced degradation. In the unsaturated zone, groundwater seepage and associated transport of dissolved leachate are estimated with one-dimensional Galerkin and upstream-weighted finite-element methods, respectively.

One-dimensional advective-dispersive transport is estimated with VADOFT based on the estimated mass flux of contaminant into the top of the soil column, and a zero concentration boundary condition at the bottom of the saturated zone. The mass flux of contaminant into the saturated zone is evaluated at the water table based on the derived concentration distribution and the Darcy velocity. The resulting mass flux from the VADOFT simulation is used as input for the AT123D model, which simulates contaminant transport through the saturated zone. It is represented as a mass flux boundary condition applied over an area representative of the facility's air-exposed surface. Because of input requirements for the AT123D code, the source area of contaminant loading to the saturated zone must be represented as rectangular in shape.

The transient nature of the flux can be represented by one of two model specifications. For most applications, the flux can be estimated by a steady-state transport analysis in the unsaturated zone. For those facilities for which closure can be reliably predicted, contaminant mass flux to the saturated zone can be represented by (1) a plug source applied at the peak level given by a transient analysis with pulse time defined by mass conservation constraints; or (2) a transient source with time-dependent flux levels interpolated from the unsaturated zone simulation results. The theory and methodology for estimating contaminant transport through the unsaturated zone are discussed in greater detail below.

4.3.2.1. FLOW OF WATER THROUGH THE UNSATURATED ZONE -- The one-dimensional downward flow of groundwater through the unsaturated zone beneath an impoundment is described by Richard's equation:

$$\frac{\delta}{\delta z} [K\, k_{rw} (\frac{\delta \psi}{\delta z} - 1)] = \eta \frac{\delta \psi}{\delta t}$$

where:

K = saturated hydraulic conductivity (m/sec)
k_{rw} = effective permeability (unitless)
ψ = pressure head (m)
z = vertical coordinate in the unsaturated zone (m)
t = time (s)

and η is the effective water storage capacity (m^{-1}), defined as

$$\eta = S_w S_s + \phi \frac{\partial S_w}{\partial t}$$

where:

S_w = the water saturation (unitless)
S_s = specific storage (m^{-1})
ϕ = the effective porosity (unitless)

This governing equation is solved subject to user-specified initial conditions and boundary conditions. The initial ($t = 0$) pressure-head profile must be specified for all depths, and the fluid flux at the top of the system and the pressure head at the bottom of the system must be assigned. All this information must be supplied to obtain a unique solution to Richards' equation.

The system as described by these equations is non-linear, because saturation depends on pressure-head, and permeability depends on saturation. To solve the governing equation, the nonlinear relationships must be defined for the soil types of interest. VADOFT allows the use of either of two functions to define the relationship between relative permeability and saturation (U.S. EPA, 1989a). These two functions, which are taken from Brooks and Corey (1966) and van Genuchten and Weirenga (1976), can be written as:

$$k_{rw} = S_e^n$$

and

$$k_{rw} = S_e^{0.5} [\ 1 - (1 - S_e^{(1/\tau)})^{\tau}]^2$$

where n and τ are empirically derived exponents. The effective water saturation (S_e) is defined as:

$$S_e = (S_w - S_{wr}) / (1 - S_{wr})$$

where S_{wr} is the residual water saturation. The empirical function in VADOFT that describes the relationship between pressure head and saturation can be written as:

$$S_e = \begin{cases} 1 / [1 + (\alpha \,|\, \psi - \psi a \,|\,)^{\beta}]^{\tau} & \text{for } \psi < \psi_a \text{, or} \\ 1 & \text{for } \psi > \psi_a \end{cases}$$

where ψ_a is the air entry pressure head, α and ß are empirically derived and τ is defined by $\tau = 1 - 1/\beta$. Statistical distributions for α, ß, τ, ϕ and S_{wr} as derived by Carsel and Parrish (1988) are given in U.S. EPA (1989b).

To represent the variably saturated soil column beneath the floor of the lagoon, the model discretizes the column into a finite-element grid consisting of a series of one-dimensional elements connected at nodal points. Elements can be assigned different properties for the simulation of flow in a heterogenous system. The model generates the grid from user-defined zones; the user defines the homogeneous properties of each zone, the zone thickness and the number of elements per zone, and the code automatically divides each zone into a series of elements of equal length. The governing equation is approximated using the Galerkin finite element method and then solved iteratively for the dependent variable (pressure-head) subject to the chosen initial and boundary conditions. Solution of the series of nonlinear simultaneous equations generated by the Galerkin scheme is accomplished by either Picard iteration, a Newton-Raphson algorithm or a modified Newton-Raphson algorithm. A detailed description of the solution scheme is found in U.S. EPA (1989a).

Once the finite-element calculation converges, the model yields estimated values for all the variables at each of the discrete nodal points. The Darcy velocity and saturation values are used as input to the unsaturated zone transport module, which is used to derive the source term for the saturated zone. From this source term, the saturated zone module determines contaminant concentrations in groundwater at the selected well location.

4.3.2.2. UNSATURATED ZONE TRANSPORT -- The governing equation (U.S. EPA, 1989a) for one-dimensional advective-dispersive transport of a dissolved species in a variably saturated soil can be written as:

$$\frac{\partial}{\partial z}\left[D\frac{\partial c}{\partial z}\right] - V_D\frac{\partial c}{\partial z} = \phi\, RF\left(\frac{\partial c}{\partial t} + \lambda c\right)$$

where:

D = the dispersion coefficient (m^2/sec)
c = the solute concentration (g/m^3)
ϕ = the porosity (unitless)
RF = the retardation factor (unitless)
V_D = the Darcy velocity (m/sec)
λ = the first-order decay coefficient (sec^{-1})

D is the dispersion coefficient, defined as:

$$D = \alpha_L V_D + \phi D^*$$

where:

α_L = longitudinal dispersivity (m)
D^* = the effective molecular diffusion coefficient (m^2/sec)

and RF is the retardation factor (unitless) defined as

$$RF = 1 + (\rho_b k_d / \phi S_w)$$

where:

ρ_b = the bulk density of wet soil (g/m^3)
k_d = the linear adsorption coefficient (m^3/g)
S_w = the water saturation (unitless)

The governing differential equation for unsaturated zone transport is solved by VADOFT subject to specified boundary and initial conditions (concentrations). The initial concentration profile must be specified for all depths below the bottom of the site, down to the bottom of the unsaturated (and saturated) zone.

In VADOFT, the one-dimensional advective-dispersive transport equation is approximated with an upstream-weighted finite-element method, which reduces to a Galerkin approximation when the weighting parameter is set to zero. The upstream weighting term is used to circumvent numerical instabilities that may occur if the problem is advective dominant, as the weighting effectively adds dispersion to the system (Huyakorn and Pinder, 1983). Linear elements are used in the spatial discretization, and time integration can be performed using either backwards or central differencing. A more detailed description of the mathematical theory and numerical approximations used in VADOFT can be found in U.S. EPA (1989a). Spatial discretization for the transport analysis parallels the discretization described above for the flow analysis. Successful model execution requires that the definition of elements and nodes be the same for the transport and flow analyses.

4.3.3. Contaminant Transport in the Saturated Zone. Two basic approaches are commonly used for estimating contaminant concentrations and travel time (or velocity) in the saturated groundwater flow system: analytical solutions and numerical modeling. Analytical solutions are relatively quick and simple to use. However, they are based on a variety of simplifying assumptions related to contaminant characteristics and the subsurface environment, and so provide only order-of-magnitude estimates of contaminant travel time and concentration. Numerical models, on the other hand, are far less restrictive with regard to simplifying assumptions; however, they typically require more data, are time consuming to set up and run, and require expensive and specialized equipment and expertise. Based on the selection criteria discussed earlier, the use of numerical models is not appropriate for the methodology proposed in this document.

Prior to selecting the AT123D model for simulating contaminant transport through the saturated zone near a surface disposal facility, the performance of that model with respect to conditions representative of surface impoundments was compared with that of a fully three-dimensional numerical model, and with that of an alternative analytical code. The numerical model used for the tests was SEFTRAN, a finite element flow and transport code, which is available from Holcomb Research Institute and has been benchmarked for HRI and the U.S. EPA Office of Solid Waste (Huyakorn et al., 1985). Its numerical solution allows contaminant migration underneath the source to be influenced by a flow field gradient and surficial infiltration effects. The alternative analytical code is similar to the analytical solutions currently used by the U.S. EPA Office of Solid Waste to evaluate solute transport in the saturated zone for generic Monte Carlo studies (Huyakorn et al., 1984; Lester et al., 1986; U.S. EPA, 1988d). It uses a partially penetrating vertical plane, or "patch source," to represent the source term for unsaturated zone transport (Lester et al., 1986). Unlike AT123D, this model represents its source term as a constant concentration boundary condition; AT123D uses a mass flux. Over a range of distances from the source, depths, and contaminant half-lives, the AT123D model compared quite favorably with the numerical code, and generally out-performed the patch source analytical solution. This methodology therefore follows U.S. EPA (1986a) in selecting the AT123D model for its simulation of the transport of contaminants through the saturated zone.

Detailed description of the AT123D model are provided by U.S. EPA (1986a) and by Yeh (1981) and will not be repeated here. In general, the model provides an analytical solution to the basic advective-dispersive transport equation. One advantage of AT123D is its flexibility: the model allows the user up to 450 options and is capable of simulating a wide variety of configurations of source release and boundary conditions. For further details concerning AT123D, the reader is referred to the landfill methodology document or to Yeh (1981). Some modifications to AT123D that are specific to the modeling of surface impoundments are briefly described below.

As mentioned above, seepage from a sludge lagoon can result in an increased local influx of water to an underlying aquifer, and can result in local elevation or mounding of the water table. The AT123D model accepts as input the flux of pure contaminant mass entering the top of the saturated zone, and does not consider the extent of the contaminant's dilution by water from the source area, or the impact of that water on groundwater flow within the saturated zone. When the vertical movement of contaminant through the unsaturated zone is due only to infiltration throughout the area, the gradient within the aquifer is a function of the water entering the saturated zone, and neglect of the diluted state of the source term may be valid. For the case of a surface impoundment, however, neglect of the extent of the contaminant's original dilution could result in non-trivial overestimation of the source concentration, leading to an overestimation of contaminant concentrations at the receptor well. Furthermore, neglect of mounding effects could lead to incorrect assumptions about the velocity of groundwater flow near the site.

The present methodology addresses these concerns with three simple adjustments to the execution of the AT123D model. First, to correct for AT123D's potential overestimation of the original concentration of contaminant at the aquifer's boundary, the mass flux estimated from VADOFT results is adjusted by a dilution factor (D_f) as follows:

$$D_f = F_a / (F_s + F_a) \tag{4-2}$$

where F_a is the volume of fluid passing through a vertical cross section of the aquifer (m^3/sec), oriented perpendicular to the direction of flow, and having a width equal to the source width and a depth equal to the saturated thickness of the aquifer. F_s is the volume of fluid leaving the impoundment (m^3/sec). In cases where seepage from the lagoon is not significant compared with the natural, regional rate of aquifer recharge, this dilution adjustment is inappropriate, and can be inactivated for program execution.

The excess water released by leakage from a surface impoundment can also result in a mounding of the water table beneath the impoundment, so that the lagoon superimposes a radial velocity field on the background or regional velocity field of groundwater flow. In other words, the horizontal velocity of water within the aquifer can be slowed up-gradient of the lagoon, and accelerated down-gradient of the site. This change in the velocity field might result in reduced time of travel for contaminants moving to receptor wells down-gradient of the impoundment site. A reduction in time of travel could in turn lead to reductions in contaminant degradation prior to human exposure. Accurate accounting of the influence of mixing and degradation would require a fully three-dimensional flow and transport model; this methodology uses a simpler approach to estimate a conservative limit to contaminant decay within the system. The limit is estimated by increasing the estimated velocity of groundwater flow to account for the maximum downgradient increase in velocity due to the source. The velocity increase can be approximated by idealizing the lagoon as a circular source, so that the rate at which seepage passes outward through a cylinder beneath the perimeter of the lagoon's floor will be $V_i = q_i r/d$, where q_i is the infiltration rate from the impoundment, r the radius of a circle of area equal to that of the impoundment, and d the aquifer depth.

In addition to increasing the expected velocity of contaminant transport through the aquifer, this superimposed velocity would also have the effect of increasing AT123D's estimate of contaminant dilution within the aquifer. This additional dilution effect must be subtracted back out of the model calculations, since the true dilution is explicitly included in the factor introduced by Equation 4-2. The model performs this calculation automatically, based on the following equation for the anti-dilution factor:

$$D_{af} = (V_v + V_h) / V_h$$

where V_v is the vertical velocity due to the source, and V_h is the regional velocity of horizontal groundwater flow.

It should be noted that the above methodology is conservative, since it overestimates the velocity beneath the source and does not allow for decreases in the superimposed velocity beyond the source. As a result, the methodology is more conservative than a three-dimensional model. In comparison with a two-dimensional cross-sectional flow and transport model, the model is more conservative beneath the source, but less conservative (but still conservative) beyond the source.

By combining the VADOFT model with AT123D, and by adjusting calculations in AT123D to accommodate the dilution and superimposed velocity described above, concentrations of a contaminant in groundwater at a receptor well (C_{gw}) can be predicted as a function of the liquid concentration of contaminant near the floor of the lagoon, the rate of seepage from the lagoon, and geohydrological characteristics of the area. It should be noted that all of the calculations described above are linear with respect to contaminant concentrations in liquid seeping from the lagoon. As will be illustrated in Section 4.7, it is convenient to perform the calculations based on the assumption of a "unit" concentration of dissolved contaminant within the facility.

The above discussion described a method for estimating concentrations of contaminant in groundwater at a specified well location down-gradient of the surface disposal site. As mentioned in Chapter 3, an additional pathway of potential exposure associated with groundwater contamination involves the discharge of contaminants from the aquifer to the surface water body, with potential subsequent exposure to humans and wildlife. Estimation of potential exposure through these pathways requires a method for estimating the expected loading of contaminants to the surface water body. This loading can occur throughout the area at which the plume of contaminated aquifer flow intersects the stream bed. Concentrations of contaminant in groundwater entering the stream can be expected to diminish toward the edge of the plume. As will be explained in Chapter 5, however, only the total mass loading of contaminant to the stream is of concern for methodologies to estimate potential exposure to humans and wildlife. With some simplifying assumptions based on a mass-balance approach, results from AT123D can be used to predict steady-state mass loadings to the surface water body.

Unless available geohydrological data suggest otherwise, it is assumed that the direction of groundwater flow is perpendicular to the stream, and that the stream intercepts the entire plume of contaminated groundwater. For contaminant species without decay (i.e. metals), it is assumed that the steady-state loading to the stream is the same as that seeping from the lagoon and loaded into the aquifer. For non-conservative species, contaminant mass will be lost to degradation during transport through the aquifer, so that the loading to the stream will be less than the predicted loading to the aquifer.

One approach to estimating the extent of this degradation would be to estimate the average time of travel for contaminant mass through the aquifer to the surface water body. This time estimate could be derived from the retarded Darcy velocity calculated for contaminant in the aquifer, together with an assumption regarding the average distance traveled (represented, perhaps, by the shortest distance from the lagoon to the surface water body). Contaminant loss to degradation could then be estimated based on assumed rates of first-order decay for the subject contaminant in the saturated zone.

A flaw in that approach is that it ignores the influence of longitudinal dispersion, which can affect the average time of travel for contaminant mass in the aquifer. An alternative approach is to use the AT123D model, constrained such that the effects of transverse and vertical dispersion are ignored. The loading of contaminant to the aquifer is represented by a rectangular prismatic source with width and length corresponding to the width and length of the lagoon, and with depth corresponding to the depth of the aquifer. If the aquifer is further specified as having a width corresponding to the width of the lagoon, then the estimated contaminant concentrations at the selected distance will approximate the expected average concentration of contaminated groundwater entering the stream. This concentration can then be multiplied by the corresponding volume of water flow in the cross-section of the aquifer under consideration, which can be estimated based on the Darcy velocity of the relevant aquifer medium. Results from AT123D (based on unit concentrations of the contaminant in the lagoon) can be used to derive a ratio between expected loading of contaminant to the stream (W_0) and the concentration of the contaminant in the lagoon (C_l). Use of this ratio for deriving criteria for surface water pathways will be further discussed in Chapter 5.

4.4. INPUT PARAMETER REQUIREMENTS

Various input parameters are needed to define the geohydrologic system through which the dissolved species is transported, and to define the behavior of individual chemical contaminants in the sludge. Reasonable worst case parameter values should be selected when deriving national (Tier 1) criteria, but more accurate, site-specific values should be used for Tier 2 calculations. Four categories of input parameters are required: (1) parameters defining the source term, (2) parameters describing characteristics of the unsaturated zone, (3) parameters describing characteristics of the saturated zone, and (4) chemical-specific parameters.

4.4.1. Source Term. The following parameters are needed to estimate the loading of contaminant to the unsaturated zone beneath a site:

1. The rate of seepage from the bottom of the surface disposal facility should be estimated for each site; alternatively, conservative default values can be used.

2. Linear adsorption coefficients for each contaminant in sludge can be taken from Appendix C of U.S. EPA (1986a) or other available literature, can be determined empirically for the sludge matrix received by a particular facility, using methods described in DeWald and Phillips (1989), or can be estimated with the CHEMEST procedures available within GEMS (U.S. EPA, 1988c).

4.4.2. Unsaturated Zone. The following parameters are needed to estimate water flow and contaminant transport through the unsaturated zone:

1. Stratigraphic variation in soil and rock types within the defined property zones should be determined from the site plan and borings.
2. Depth to groundwater (without consideration of local mounding caused by seepage) should be determined from the site plan and borings.
3. Saturated hydraulic conductivity and effective porosity of various soil and rock types should be calculated from field measurements, or can be taken from Freeze and Cherry, 1979) or other sources.
4. Empirical parameters needed to define the effective permeability-saturation relationship and saturation-pressure head relationship or sets of data points describing the effective permeability-saturation curves and the saturation-pressure head curves for each soil and rock type being modeled are available from U.S. EPA (1989b), or Carsel and Parrish, (1988).
5. Distributions of residual saturations and air entry pressure heads for each soil and rock type are available from U.S. EPA (1989b).
6. In the absence of site-specific values, longitudinal dispersivity can be estimated as one-tenth the thickness of the vadose zone (Carsel and Parrish, 1988).
7. Linear adsorption coefficients and decay coefficients for each material zone can be taken from Appendix C of U.S. EPA (1986a); alternatively, absorption coefficients can be derived empirically, using methods described in Appendix B of that reference, or can be estimated with the CHEMEST procedures in GEMS. If desired, and if sufficient data are available, decay coefficients can also be used to represent pseudo-decay processes for heavy metals, including the formation of stable precipitates with carbonates, sulfates, hydroxides, etc.

4.4.3. Saturated Zone. The following input parameters are needed to simulate contaminant transport through the saturated zone:

1. The geometry of the region of interest including thickness of the saturated zone and lateral extent of the aquifer should be determined from the facility site plan, or from geologic survey reports.
2. The areal extent of the source can be determined from the facility site plan.
3. Effective porosity and bulk density of the aquifer material can be determined from soil measurements, or taken from Appendix C of U.S. EPA (1986a).

4. Linear adsorption coefficients and decay coefficients for each material zone can be taken from Appendix C of U.S. EPA (1986a); alternatively, adsorption coefficients can be derived empirically (U.S. EPA, 1986a) or other available literature, or can be estimated with the CHEMEST procedures in GEMS.

5. Hydraulic conductivity can be determined from field tests or taken from Appendix C of U.S. EPA (1986a).

6. Longitudinal, transverse and vertical dispersivities should be calibrated against field data, extrapolated from values calculated for similar aquifer systems, or drawn from ranges supplied in Yeh (1981), calculated with methods presented in U.S. EPA (1986d).

7. Mass flux as determined by the unsaturated zone analysis.

8. Distance from the impoundment to the property boundary or surface water can be determined from the site plan.

4.5. HEALTH AND ENVIRONMENTAL EFFECTS

Toxic pollutants that leach from surface disposal sites into groundwater can cause adverse health effects to nearby residents who use wells as a source of drinking water. Wildlife could be exposed directly to the surface of the lagoon, but are unlikely to be exposed to leached contaminants until the contaminated groundwater discharges into streams or other surface water bodies. Potential contamination of surface water through this pathway, and its resulting risks to human health and wildlife, will be discussed in Chapter 5.

The first step in deriving sludge criteria to prevent risks to human health from contaminated groundwater is to determine a reference water concentration (RWC) for each sludge contaminant of concern. The procedure for determining the RWC differs according to whether the pollutant acts by a threshold or non-threshold mechanism of toxicity. Each of these two types of toxicant will now be discussed.

4.5.1. Threshold-Acting Toxicants. Threshold-acting toxicants are those for which a dose can be identified below which no adverse effects are expected to occur. The Agency assumes that all non-carcinogenic chemicals act according to threshold mechanisms. The RWC is derived as follows for threshold-acting toxicants:

$$RWC = [(RfD\ BW\ RE^{-1}) - TBI] / I_w$$

where:

RWC = reference water concentration (mg/*l*)
RfD = reference dose (mg/kg/day)
BW = human body weight (kg)
RE = relative effectiveness of ingestion exposure (unitless)
TBI = total background intake rate of contaminant from all other sources of exposure mg/day)
I_w = total water ingestion rate (*l*/day)

The definition and derivation of each of the parameters used to estimate RWC for threshold-acting toxicants are further discussed in the following sections.

4.5.1.1. **REFERENCE DOSE (RfD)** -- For threshold-acting toxicants, the U.S. EPA establishes a "Reference Dose" (RfD), defined as the lifetime dose of a chemical that is likely to be without appreciable risk of deleterious effects. The RfD is expressed in milligrams of dose per kilogram of body weight. Procedures for estimating reference doses from various types of toxicological data were outlined in U.S. EPA (1980c), and more recently in U.S. EPA (1990). Values of RfD for noncarcinogenic or systemic toxicity have been derived by several groups within the Agency. An intra-Agency Reference Dose Work Group now verifies these values by consensus for use by the various Agency programs. Most of the non-carcinogenic chemicals that are currently candidates for sludge criteria for surface disposal are included in the Agency's verified RfDs, and thus no new effort will be required to establish RfDs for deriving sludge criteria. For any chemicals not so listed, RfD values should be derived according to established Agency procedures (U.S. EPA, 1990).

4.5.1.2. **HUMAN BODY WEIGHT (BW)** -- Values for human body weight vary widely among individuals according to age and sex. The variations in mean body weight with age and sex for the U.S. population are illustrated in Table 4-4. The choice of values for use in the risk assessment depends on the individual defined as the most exposed individual, or MEI, which in turn depends on exposure and susceptibility to adverse effects. In cases where effects on which the RfD is based may occur after cumulative lifetime exposure, it would be reasonable to base derivation of criteria on adult values of body weight. In circumstances where effects have a shorter latency, or where children are known to be at special risk, it may be more appropriate to base the derivation of

TABLE 4-4

Water Ingestion and Body Weight by Age-Sex Group in the United States*

Age-Sex Group	Mean Water Ingestion (ml/day)	Median Body Weight (kg)	Water Ingestion per Unit Body Weight (ml/kg-day)
6-11 months	308	8.8	35.1
2 years	436	13.5	32.2
14-16 years, females	587	51.3	11.4
14--16 years, males	732	54.2	13.5
25-30 years, females	896	58.5	15.3
25-30 years, males	1050	67.6	15.5
60-65 years, females	1157	67.6	17.1
60-65 years, males	1232	73.9	16.7

*Source: U.S. EPA, 1986a

criteria on the body weight of toddlers or infants. The approach used in the derivation of Maximum Contaminant Level Goals (MCLGs) by the U.S. EPA Office of Drinking Water is to assume an adult body weight of 70 kilograms.

4.5.1.3. **TOTAL WATER INTAKE RATE (I_w)** -- Total daily ingestion of water varies widely among individuals according to age and sex. Table 4-5 shows the variation of adult drinking water intake within and among several studies. Mean intakes in New Zealand, Great Britain, the Netherlands and Canada varied from 0.96 to 1.30 liters per day. More recently, U.S. EPA (1989e) has presented conservative and average water consumption values of 2 *l*/day and 1.4 *l*/day, respectively. The choice of values for use in the risk assessment depends on the individual defined as the MEI, which in turn depends on exposure and susceptibility to adverse effects. As shown in Table 4-4, the total water intake on a body-weight basis is substantially higher for infants and toddlers than for adults. Therefore, infants and toddlers would be at greater risk of exceeding the RfD when exposure is by drinking water. However, in cases where effects on which the RfD is based may occur after cumulative lifetime exposure, it would be reasonable to base derivation of criteria on adult values of water intake. In circumstances where effects have a shorter latency, or where children are known to be at special risk, it may be more appropriate to base the derivation of criteria on the daily consumption rate of toddlers or infants. The approach used in the derivation of Maximum Contaminant Level Goals (MCLGs) by the U.S. EPA Office of Drinking Water is to assume a total water ingestion rate (I_w) of 2.0 liters per day for adults. This value exceeds the 90th percentile estimates from the studies presented in Table 4-5 and thus represents a relatively high estimate of exposure, although the total range can extend well above this value, at least on a given sampling day. On a body-weight basis, however, this value represents an intake of 28.6 ml/kg/day, a value lower than the mean intake for infants and toddlers. Therefore, water intake values for children should be used in cases where children are at greater risk than adults. For example, the MCLG for lead, a toxicant known to threaten children, was based on exposure for infants rather than adults (U.S. EPA, 1985e).

TABLE 4-5*

Daily Intakes of Drinking Water by Adults
(l/day)

Mean ± sd	Range	Percentiles 10th	 50th	 90th	n	Method	Country
1.25 ± 0.39	0.26 - 2.80	0.42	1.26	1.90	109	Duplicate sampling	New Zealand
0.96 ± 0.57	0.00 - 6.47	0.40	0.87	1.64	1960	24-hour recall	New Zealand
1.08		0.50	—	1.68	2596	Diary	Great Britain
1.26					1472		Netherlands
1.30					342		Canada

*Source: Gillies and Paulin, 1983

4.5.1.4. TOTAL BACKGROUND INGESTION RATE (TBI) -- Sources of exposure other than sludge reuse or disposal practice may exist, and total exposure from all sources should be maintained below the RfD. Other sources of exposure include background levels (whether natural or anthropogenic) in drinking water, food or air. Other types of exposure that are due to occupation or habits such as smoking might also be included, depending upon data availability and regulatory policy. These exposures are summed to estimate the total background ingestion rate.

Data for estimating background exposure usually are derived from analytical surveys of surface, ground or tap water, from Food and Drug Administration market basket surveys and from air-monitoring data. These surveys may report estimates of central tendency, percentiles or ranges. Estimates of the TBI may be based on either estimates of central tendency or upper-bound estimates. Data chosen to represent the TBI of the most exposed individual should be consistent with other characteristics of the MEI. For example, if the body weight of a child is used in the derivation of criteria, then the TBI used should be based on the TBI estimated for children. Background data reported as concentrations in air, food or water can be converted to values for background intake by using appropriate estimates of ingestion or inhalation rates. Data reported for adults can be used to estimate intakes for children, using adjustments based on relative food, water or air intakes.

The TBI is the sum of all possible background exposures except those exposures resulting from the sludge disposal practice. The "effective" TBI is the sum of all background exposures weighted by the relative effectiveness of the route through which each exposure occurs. The effective TBI can be derived as follows:

$$\mathrm{TBI} = \frac{\mathrm{BI}_{food}}{\mathrm{RE}_{food}} + \frac{\mathrm{BI}_{water}}{\mathrm{RE}_{water}} + \frac{\mathrm{BI}_{air}}{\mathrm{RE}_{air}} + \ldots + \frac{\mathrm{BI}_{n}}{\mathrm{RE}_{n}}$$

where:

TBI = total background intake rate of contaminant from all other sources of exposure (mg/day)

BI = background intake of pollutant from a given exposure route, indicated by the subscript (mg/day)

RE = relative effectiveness, with respect to ingestion exposure, of the exposure route indicated by the subscript

When TBI is subtracted from the weight-adjusted RfD, the remainder (after adjusting for RE) defines the increment that can result from sludge disposal without exceeding the RfD. The magnitude of the impact on the exposed population resulting from sludge-related exposures that exceed this increment will depend on the data used to determine the TBI. If the TBI is based on upper-bound data, such as the 95th percentile data, then sludge-related exposures exceeding this increment would imply that about 5% of the exposed population may approach or exceed the RfD. If the TBI is based on an estimate of central tendency, such as the median, then about 50% of the exposed population would be expected to approach or exceed the RfD.

4.5.1.5. RELATIVE EFFECTIVENESS OF EXPOSURE (RE) -- The relative effectiveness (RE) of exposure shows the relative toxicological effectiveness of an exposure by a given route when compared with another route. The value of RE may reflect observed or estimated differences in absorption between the inhalation and ingestion routes that can significantly affect the quantity of the pollutant that reaches the target tissue, the time required for the contaminant to reach the target tissue and the degree and duration of the effect. The RE factor may also reflect differences in the occurrence of critical toxicological effects at the point of entry. For example, carbon tetrachloride and chloroform were estimated to be 40% and 65% as effective, respectively, by the inhalation route as by the ingestion route (U.S. EPA, 1984a,b). In addition to route differences, RE can also reflect differences in bioavailability resulting from the exposure matrix. For example, absorption of nickel via the inhalation route has been estimated to be 5 times the absorption when ingested in the diet (U.S. EPA, 1985a). The presence of food in the gastrointestinal tract may delay absorption and reduce the availability of orally administered compounds, as demonstrated for the halocarbons (NRC, 1986).

Since exposure from drinking contaminated water is through oral ingestion, the RE factors applied in the above equations represent relative effectiveness of exposure routes and matrices from which the RfD was derived when compared with exposure via food or drinking water. In these equations, the RE factors show the relative effectiveness, with respect to the oral route, of each background exposure route and matrix.

An RE factor should be applied only where well-documented, well-referenced information is available on the contaminant's observed relative effectiveness. When such information is not available, the RE is assumed to be equal to 1.

4.5.2. Carcinogens. For carcinogenic chemicals, the Agency considers the excess risk of cancer to be linearly related to dose, except at high dose levels (U.S. EPA, 1986c). The threshold assumption, therefore, is not applicable to carcinogens, since incremental carcinogenic risk from the exposure diminishes with dose but does not become zero until dose becomes zero.

The decision whether to treat a chemical as a threshold-acting or carcinogenic agent depends on the weight-of-evidence that the chemical is carcinogenic in humans. Methods for classifying chemicals as to their weight-of-evidence have been described in U.S. EPA (1986c), and most of the chemicals that are candidates for sludge criteria development have been classified in Health Assessment Documents or other reports prepared by EPA's Office of Health and Environmental Assessment (OHEA), or in connection with the development of Maximum Contaminant Level Goals (MCLGs). To derive values of the adjusted reference intake, a decision must be made as to which classifications constitute sufficient evidence for using a quantitative risk assessment on the presumption of carcinogenicity. Chemicals are classified as follows: an "A" designation is given to a known human carcinogen; a "B" classification is given to a probable human carcinogen; a possible human carcinogen is designated as "C"; a "D" chemical is "not classifiable because of inadequate animal and human data"; and an "E" designation is given to chemicals for which there is evidence of non-carcinogenicity in humans. In general, "A" and "B" chemicals are viewed as important hazard levels for public health concerns, while "D" and "E" chemicals are not. "C" chemicals have received varying treatment. For example, the regulatory action on lindane, classified by the Human Health Assessment Group (HHAG) of the U.S. EPA as a "B2-C" (that is, between the lower range of the B category and the C category), has been based on both carcinogenic risk, (U.S. EPA, 1980b) and threshold effects (U.S. EPA, 1985e). Table 4-6 gives an illustration of the EPA classification system based on available weight-of-evidence.

The use of weight-of-evidence classification must also be coupled with information regarding the route by which the chemical demonstrates carcinogenicity. For example, some forms of nickel

TABLE 4-6

Illustrative Categorization of Evidence Based on Animal and Human Data[a,b]

Human Evidence	Animal Evidence				
	Sufficient	Limited	Inadequate	No Data	No Evidence
Sufficient	A	A	A	A	A
Limited	B1	B1	B1	B1	B1
Inadequate	B2	C	D	D	D
No data	B2	C	D	D	E
No evidence	B2	C	D	D	E

[a] Source: U.S. EPA, 1986c

[b] The above assignments are presented for illustrative purposes. There may be nuances in the classification of both animal and human data indicating that different categorizations than those given in the table should be assigned. Furthermore, these assignments are tentative and may be modified by ancillary evidence. In this regard all relevant information should be evaluated to determine if the designation of the overall weight-of-evidence needs to be modified. Relevant factors to be included along with the tumor data from human and animal studies include structure-activity relationships, short-term test findings, results of appropriate physiological, biochemical and toxicological observations, and comparative metabolism and pharmacokinetic studies. The nature of these findings may cause an adjustment of the overall categorization of the weight-of-evidence.

have been shown to be carcinogenic by inhalation but not by ingestion (U.S. EPA, 1985a). Similarly, arsenic has been shown to cause carcinogenic effects when certain inorganic forms are ingested in water, but no carcinogenic potential has been demonstrated for the organic forms of arsenic commonly present in many foods. The issue of whether to treat a chemical as carcinogenic by ingestion is controversial for many chemicals.

For pollutants assessed as carcinogenic (non-threshold acting) agents, the reference water concentration (RWC, in mg/*l*) in groundwater used for drinking is derived as follows:

$$RWC = (RL\ BW)/(q_1^*\ RE\ I_w)$$

where:

RWC = reference water concentration (mg/*l*)
RL = risk level (unitless)
BW = human body weight (kg)
q_1^* = human cancer potency $(mg/kg\text{-}day)^{-1}$ or (kg-day/mg)
RE = relative effectiveness of ingestion exposure (unitless)
I_w = total water ingestion rate (*l*/day)

The RWC, in the case of carcinogens, is thought to be protective, because the estimate of carcinogenicity is an upper limit value. The parameters BW, RE and I_w are the same for carcinogens and for threshold-acting toxicants; these parameters are defined and described in Section 4.5.1. The definition and derivation of q_1^* and RL are discussed in the following sections.

4.5.2.1. HUMAN CANCER POTENCY (q_1^*) -- For most carcinogenic chemicals, the linearized multistage model is recommended for estimating human cancer potency from animal data (U.S. EPA, 1986c). When epidemiologic data are available, potency is estimated based on the observed relative risk in exposed vs. unexposed individuals, and on the magnitude of exposure. Guidelines for the use of these procedures have been presented in U.S. EPA (1980c) and U.S. EPA (1985e) and in each of a series of Health Assessment Documents prepared by OHEA (such as U.S. EPA, 1985b). The true potency value is considered unlikely to be above the upper-bound estimate of the slope of the dose-response curve in the low-dose range, and is expressed in terms of risk per dose, where dose is in units of mg-day/kg. Thus, the q_1^* has units of $(mg\text{-}day/kg)^{-1}$ or kg/mg-day.

OHEA has derived potency estimates for each of the potentially carcinogenic chemicals that are currently candidates for sludge criteria. Therefore, no new effort will be required to develop potency estimates to derive the sludge criteria.

4.5.2.2. RISK LEVEL (RL) -- Since by definition no "safe" level exists for exposure to non-threshold agents, values of RWC are calculated to reflect various levels of cancer risk. If the RL is set at zero, then the RWC will be zero. If RL is set at 10^{-6}, the RWC will be the concentration that, for lifetime exposure, is calculated to have an upper-bound cancer risk of 1 case in 1 million exposed individuals. This risk level refers to excess cancer risk, that is, risk over and above the background cancer risk in unexposed individuals. By varying RL, the RWC may be calculated for any level of risk in the low-dose region (i.e. for RL less than 10^{-2}). Specification of a given risk level on which to base regulations is a matter of policy, as is the usefulness of the upper bound concept in risk derivation. Therefore, it is common practice to derive criteria representing several levels of upper bound risk without specifying any risk level as "acceptable."

4.6. DERIVING CRITERIA

Section 4.3.1 described a method for predicting contaminant concentrations in seepage as a function of the dry-weight concentration of contaminant in sludge received by an impoundment. Sections 4.3.2, 4.3.3, and 4.3.4 described calculations that used the expected contaminant concentration in seepage, together with the quantity of seepage, to predict expected contaminant concentrations in groundwater at the property boundary. Section 4.5 presented methods for deriving reference water concentrations (RWC) for groundwater. The final step in criteria derivation is to combine these results to link dry-weight contaminant concentrations in sludge to expected well-water concentrations and health effects, or conversely, to link reference water criteria to maximum allowable dry-weight concentrations. In this context, it is useful to define a "source-receptor ratio" or SRR_{GW}, that represents the ratio of contaminant concentration in well-water (mg/l) to dry-weight concentration of contaminant in sludge received or accumulated in a facility (mg/kg). Since all of the calculations described in Sections 4.3 are linear with respect to contaminant concentrations, SRR_{GW} (kg/l) can be easily derived by combining results from the calculations and model results described above:

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = C_{gw}/N \quad (4\text{-}3)$$

where:

SRR_{GW} = the source-receptor ratio (kg/l)
C_{gw} = predicted concentration of contaminant in groundwater at a receptor well location (mg/l)
C_l = predicted concentration of contaminant in liquid phase near the floor of the lagoon (mg/l)
N = dry-weight concentration of contaminant in sludge received by or accumulated in the facility (mg/kg)

This ratio can then be used to estimate the maximum allowable concentrations of contaminant in sludge received by a facility:

$$N_{max} = RWC/SRR_{GW} \quad (4\text{-}4)$$

where:

N_{max} = the maximum allowable dry-weight concentration of contaminant to be received by or accumulated in the impoundment (mg/kg)
RWC = reference water concentration (mg/l)

If background groundwater concentration levels for the contaminants of interest are measurable, these should be incorporated into the calculation:

$$N_{max} = (RWC - C_b)/SRR_{GW}$$

where C_b is the background concentration for the contaminant in groundwater. Average or reasonable worst-case background concentrations for groundwater in the United States can be used to derive national criteria.

4.7. SAMPLE CALCULATIONS

Sample calculations are provided for Tier 2 analysis of three actual surface disposal facilities: a wastewater treatment and sludge storage facility in New Hampshire, a sludge storage facility in Oklahoma and a sludge disposal facility in Oregon. These three sites were chosen for sample calculations because they were thought to represent the considerable variability in facility types that must be considered for actual application of the methodologies presented in this document. Two contaminants, lead and benzene, will be considered for the sample calculations. Lead was chosen to represent the use of the methodology for deriving criteria for a heavy metal, and because lead concentrations may be limiting in many actual applications. Benzene has been chosen to illustrate the application of the methodology for volatile organic contaminants; its volatility will be of special interest in sample calculations presented in Chapters 5 and 6. It will be assumed that all three facilities fail national (Tier 1) criteria, so that Tier 2 calculations can be described.

Tier 2 calculations rely on computer models that estimate expected contaminant concentrations in groundwater as a function of liquid contaminant concentrations in sludge entering the impoundments. Although the computer model is designed to estimate concentrations in groundwater, results can also be used for "reverse" calculations of sludge concentrations as a function of RWC, since ultimate estimates of groundwater concentrations are in all cases a linear function of concentrations in the impoundments, which are in turn assumed to be a linear function of dry weight sludge concentrations. For simplicity, site-specific simulations of contaminant transport in groundwater derived for these sample calculations will be based on unit concentrations of contaminant in seepage beneath the facility to be considered. Results from a simulation based on unit concentrations will be used to derive a source-receptor ratio (SRR_{GW}) that describes the ratio of contaminant concentrations in groundwater at the receptor well to contaminant concentrations in sludge received by the lagoons. From this ratio, criteria for maximum allowable sludge concentrations at these sites will be derived.

Execution of the VADOFT and AT123D models for groundwater simulations requires values for numerous input parameters descriptive of the site under consideration. For the sample calculations presented here, some of the required data were taken from site visits, interviews with plant operators, inspection of site plans and core sample results. Other data were derived from literature values judged to be representative of site conditions; many of the ground material parameters have been estimated from data obtained from general data for soils of similar characteristics (Carsel and Parrish, 1988). The following sections briefly discuss input parameters, model results and criteria derivation for each of the three surface disposal sites considered.

4.7.1. Analysis of Exposure for the Most Exposed Individual.

4.7.1.1. ANTRIM, NEW HAMPSHIRE -- The Antrim wastewater treatment plant is a small facility with a design capacity of 0.23 million gallons per day (MGD), that uses aerated lagoons as its method of wastewater treatment. Unlike the other impoundments to be considered in sample calculations, those in the Antrim facility receive wastewater, not sludge. Since sludge remains in the bottom of the lagoon for years or decades between emptying, however, these wastewater treatment lagoons have been included in the sample calculations presented by this document.

The Antrim facility contains three lagoons lined with polyvinyl chloride (PVC): one with top dimensions of 125 by 232 feet (2700 m^2 area) and two smaller lagoons, each with dimensions of 101 by 125 feet each (1173 m^2 area). The facility was built into the side of a hill, on a slope leading to the Contoocook River (about 100 meters from the site). Each lagoon is about 10 feet deep (3 meters) and surrounded by about 1 meter of vertical freeboard. Observation wells near the perimeter of the lagoons allow monitoring of groundwater level. The water table occasionally rises to a level above the bottom of the lagoons; a sump pump is available to lower the water table beneath the facility if groundwater threatens to damage the liner of a partially filled lagoon.

After 8 years of wastewater treatment, the floors of the impoundments have collected about 14-18 inches (36-46 cm) of sludge. The facility operator does not plan to remove the sludge until it begins to interfere with wastewater treatment; that condition is not expected for another 20 years or more.

Both Tier 1 and Tier 2 calculations require the derivation of a reference water concentration, or RWC. Since benzene is a carcinogen with an established potency value, the RWC (in mg/day) for drinking is derived as follows:

$$\mathrm{RWC} = (\mathrm{RL}\ \mathrm{BW}) / (q_1^{*}\ \mathrm{RE}\ I_w)$$

For this sample calculation, a risk level of 10^{-6} is selected. With an assumed body weight of 70 kg, an average rate of total water ingestion of 2 liters per day, a relative effectiveness of 1 for benzene through ingestion, and a human cancer potency (q_1^*) of 2.9×10^{-2} (kg-day/mg), the RWC for benzene is calculated to be:

$$RWC = [(10^{-6})(70)] / [(2.9 \times 10^{-2})(1)(2)] = 1.2 \times 10^{-3} \text{ (mg/}l\text{)}$$

No human cancer potency slope is yet available for lead, so it must be considered to be a threshold acting toxicant to estimate a reference water concentration. Sample calculations will be based on the proposed drinking water standard for lead, or 0.005 mg/*l* (U.S. EPA, 1988e).

Tier 2 calculations involve the use of computer models to predict the extent to which contaminant concentrations in seepage beneath the impoundment will be attenuated as a result of underground transport to a receptor well. The calculations begin by using the VADOFT model to calculate estimated concentrations of contaminant entering the saturated zone beneath the site. These results are then taken as input to the AT123D model to derive estimates of expected concentrations at a receptor well.

Records of test borings beneath the Antrim site reveal top layers of silty till followed by layers of sand or sandy gravel. For the groundwater simulations performed for these sample calculations, it is assumed that both the unsaturated and saturated zones beneath the site consist of sand. The sample calculations presented in this document ignore the effects of the PVC liner, and are therefore likely to overestimate actual seepage to groundwater. Rates of seepage from the impoundment are approximated by typical rates of seepage for municipal wastewater lagoons built on sandy material. U.S. EPA (1987b) reports that water seepage from such facilities is approximately 1.2 inches/day, or about 1.3×10^{-3} m/hour (1.3 kg/m^2-hour). This value is used for model calculations.

Input parameters values used for the VADOFT simulation are presented in Table 4-7. For both flow and transport calculations, it is assumed that the unsaturated zone beneath the facility consists of a sandy layer about 17 meters thick with a hydraulic conductivity of 0.3 m/hour. For

TABLE 4-7

Input Parameters for VADOFT Simulation of Flow and Contaminant Transport Through the Unsaturated Zone
Antrim, New Hampshire

Parameter	Value	(Units)
Source area	5040	(m^2)
Distance to bottom of saturated zone	17	(m)
Input Parameters for Flow Calculations		
Flux at Top Node	1.3	(l/m^2-hr)
Head at bottom node	15	(m)
Hydraulic conductivity	0.30	(m/hr)
Effective porosity	0.43	(unitless)
Specific Storage	0	(unitless)
Residual water saturation	0.105	(unitless)
Power index (N)	-1.0	(unitless)
Leading coefficient	14.5	(unitless)
Power index (β)	2.68	(unitless)
Power index (γ)	0.62	(unitless)
Input Parameters for Transport Calculations		
Concentration of Benzene, Lead	1	(mg/l)
Mass flux of contaminant to top node	1.3×10^{-6}	(kg/m^2-hr)
Head at bottom node	0	(m)
Longitudinal dispersivity	1.0	(m)
Effective porosity	0.43	(unitless)
Retardation coefficient	1.28	(unitless)
Molecular diffusion coefficient	0	(unitless)
Default Darcy velocity	0	(m/hr)
Default water saturation	1.0	(unitless)
Solute decay constant (benzene)	2.64×10^{-4}	(hr^{-1})
Solute decay constant (lead)	0	(hr^{-1})

benzene, the retardation factor is calculated based on an assumed distribution coefficient (k_d) of 0.08 l/kg (Hounslow, 1983), a bulk density of 1.51 kg/l, and a porosity of 0.43:

$$RF = 1 + (k_d\rho_b/\phi) = 1 + [(0.08)(1.51)/(0.43)] = 1.28$$

Baes et al. (1984) reports that k_d values for lead range from 4.5-7640 l/kg. For these sample calculations, a value of 234 l/kg (appropriate for sand) is taken from Appendix C of U.S. EPA, 1986a, so

$$RF = 1 + (k_d\rho_b/\phi) = 1 + [(234)(1.51)/(0.43)] = 823$$

Benzene is assumed to have a decay coefficient of $2.64\text{x}10^{-4}$ (hr^{-1}) in the unsaturated zone; lead is assumed to have a decay coefficient of zero.

The flux of contaminant mass entering the unsaturated zone from the impoundment can be described by the rate of seepage (in kg/m^2-hour) multiplied by the concentration of contaminant in the seepage (in kg/kg). These calculations assume that the seepage contains contaminant at a unit concentration of 1 mg/l, so that each liter of seepage contains $1\text{x}10^{-6}$ kg of contaminant. Contaminant flux into the unsaturated zone is then equal to the rate of seepage (1.3 kg/m^2-hour) multiplied by the contaminant concentration (in kg/kg) to yield 1.3×10^{-6} kg/m^2-hour. For VADOFT and AT123D calculations, the three lagoons in Antrim are modeled in aggregate and are idealized as a single square lagoon with the appropriate total surface area (5040 m^2).

VADOFT simulates vertical flow and contaminant transport from the floor of the impoundment to the bottom of the saturated zone. The total thickness of the soil layers to be simulated by VADOFT is therefore equal to the sum of the thicknesses of the unsaturated and saturated zones. For Antrim, it is assumed that the depth to groundwater is about 2 meters and that the aquifer thickness is 15 meters, resulting in a total distance of 17 meters from the floor of the lagoon to the bottom of the saturated zone.

Based on these and other parameter values listed in Table 4-7, the VADOFT model produces the results contained in Table 4-8. As can be seen from the table, VADOFT predicts that the water table beneath the site will be elevated an additional 0.3 meters as a result of seepage from the impoundment. The flux of contaminant mass leaving the unsaturated zone is estimated to be 1.2×10^{-6} kg/m^2-hour for benzene. This value, multiplied by the area of the site (5040 m^2) yields the total rate of benzene release to the saturated zone, or 5.9×10^{-3} kg/hour. For lead, the mass flux is 1.3×10^{-6} kg/m^2-hour (the same as the flux of lead in seepage from the lagoon), which yields a total mass release rate of 6.4×10^{-3} kg/hour. These values are used as input for the simulation of contaminant transport through the saturated zone, as performed by AT123D.

Table 4-9 lists values for input parameters required for the AT123D simulation. As can be seen from the table, the waste release rate for both benzene and lead is taken from results generated by the VADOFT model. Values for the beginning and ending of the x-source and y-source locations represent the dimensions of an idealized square impoundment with area defined by the total area of the three lagoons at the Antrim facility. Other values are selected to be representative of an aquifer consisting of sand. The superimposed velocity term (1.7×10^{-3} m/hr) represents a conservative estimate of the increment to groundwater velocity caused by seepage from the impoundment. Groundwater concentrations at a well location 152 meters downgradient of the site (or 221 meters from the far edge of the lagoon) were assumed to reflect maximum concentrations likely to be encountered by a most exposed individual. For estimating potential exposure through the drinking water pathway, it is assumed that groundwater does not flow in the direction of the Contoocook River, which is located on the property boundary at a lesser distance.

Results from the AT123D simulation are presented in Table 4-10. The table shows that steady-state concentrations of 4.2×10^{-3} mg/*l* for benzene and 0.38 mg/*l* for lead are predicted at the receptor well, based on 1 mg/*l* unit concentrations of each contaminant in the impoundment. These concentrations can be used to generate ratios of well concentrations to impoundment concentrations for each of the two contaminants. For benzene:

$$C_{gw} / C_l = 0.0042 / 1 = 0.0042$$

TABLE 4-8

Results from VADOFT Execution
Antrim, New Hampshire

Parameter	Value	(Units)
Results from Flow Simulation		
Elevation of water table	0.3	(m)
Results from Simulation of Benzene Transport		
Net dispersive flux	-2.1×10^{-7}	(kg/m^2-hr)
Net advective flux	1.2×10^{-6}	(kg/m^2-hr)
Cumulative mass decay	1.0×10^{-6}	(kg/m^2-hr)
Cumulative mass inflow	1.3×10^{-6}	(kg/m^2-hr)
Cumulative mass outflow	-2.7×10^{-7}	(kg/m^2-hr)
Mass flux	1.2×10^{-6}	(kg/m^2-hr)
Waste release rate	5.9×10^{-3}	(kg/hr)
Results from Simulation of Lead Transport		
Net dispersive flux	-1.3×10^{-6}	(kg/m^2-hr)
Net advective flux	1.3×10^{-6}	(kg/m^2-hr)
Cumulative mass decay	0	(kg/m^2-hr)
Cumulative mass inflow	1.3×10^{-6}	(kg/m^2-hr)
Cumulative mass outflow	-1.3×10^{-6}	(kg/m^2-hr)
Mass flux	1.3×10^{-6}	(kg/m^2-hr)
Waste release rate	6.4×10^{-3}	(kg/hr)

TABLE 4-9

Input Parameters for AT123D Simulation of Contaminant Transport Through the Saturated Zone
Antrim, New Hampshire

Parameter	Value	(Units)
Distance to receptor well	221	(m)
Aquifer width	infinite	(m)
Aquifer depth	15	(m)
Begin point of x-source location	0	(m)
End point of x-source location	71	(m)
Begin point of y-source location	-35.5	(m)
End point of y-source location	35.5	(m)
Porosity	0.43	(unitless)
Hydraulic conductivity	7.13	(m/hr)
Hydraulic gradient	0.01	(unitless)
Longitudinal dispersivity	15.3	(m)
Lateral dispersivity	5.1	(m)
Vertical dispersivity	1.0	(m)
Distribution coefficient (benzene)	0.08	(l/kg)
Distribution coefficient (lead)	234	(l/kg)
Decay constant (benzene)	2.6×10^{-4}	(hr^{-1})
Decay constant (lead)	0	(hr^{-1})
Density of soil	1.51	(kg/l)
Density of water	1.0	(kg/l)
Waste release rate (benzene)*	5.9×10^{-3}	(kg/hr)
Waste release rate (lead)*	6.4×10^{-3}	(kg/hr)
Superimposed velocity term	1.7×10^{-3}	(m/hr)

*Generated by VADOFT component of model code.

TABLE 4-10

Results from AT123D Simulation of Contaminant Transport Through the Saturated Zone Antrim, New Hampshire

Parameter	Value	(Units)
Results from Simulation of Benzene Transport		
Retardation factor	1.28	(unitless)
Retarded Darcy velocity	8.5×10^{-3}	(m/hr)
Retarded longitudinal dispersion coefficient	0.13	(m^2/hr)
Retarded lateral dispersion coefficient	0.04	(m^2/hr)
Retarded vertical dispersion coefficient	8.5×10^{-3}	(m^2/hr)
Steady state concentration at receptor well	4.2×10^{-3}	(mg/l)
Results from Simulation of Lead Transport		
Retardation factor	823	(unitless)
Retarded Darcy velocity	1.3×10^{-5}	(m/hr)
Retarded longitudinal dispersion coefficient	2.0×10^{-4}	(m^2/hr)
Retarded lateral dispersion coefficient	6.7×10^{-5}	(m^2/hr)
Retarded vertical dispersion coefficient	1.3×10^{-5}	(m^2/hr)
Steady state concentration at receptor well	0.38	(mg/l)

In other words, for each 1 mg/*l* of contaminant in seepage beneath the impoundment, 0.0042 mg/*l* of benzene are expected in the aquifer 500 feet (152 m) downgradient from the site. From Equation 4-1, it follows for benzene in a sludge layer with 10% solids:

$$C_l/N = 1/(k_d + r_{sl}^{-1}) = 1/(37+10) = 0.021 \text{ kg}/l$$

Then from Equation 4-3:

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = (0.0042)(0.021 \text{ kg}/l) = 8.9 \times 10^{-5} \text{ kg}/l$$

and from Equation 4-4:

$$N_{max} = RWC/SRR_{GW} = (1.2\text{x}10^{-3} \text{ mg}/l)/(8.9\text{x}10^{-5} \text{ kg}/l) = 13 \text{ mg/kg}$$

Similar calculations can be performed for lead:

$$C_{gw}/C_l = 0.38 / 1 = 0.38$$

$$C_l/N = 1/(k_d + r_{sl}^{-1}) = 1/(234+10) = 4.1\text{x}10^{-3} \text{ kg}/l$$

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = (0.38)(4.1\text{x}10^{-3}) = 1.6\text{x}10^{-3} \text{ kg}/l$$

$$N_{max} = RWC / SRR_{GW} = (0.005 \text{ mg}/l)/(1.6\text{x}10^{-3} \text{ kg}/l) = 3.1 \text{ mg/kg}$$

If actual concentrations in the sludge exceed these criteria, then long-term storage of sludge in the lagoons would not be allowed.

An additional pathway of potential human and nonhuman exposure to contaminants from surface disposal facilities involves the discharge of contaminants from groundwater to surface water, with subsequent exposure to sludge contaminants for aquatic life, or to humans, through ingestion of the water or of fish caught in the contaminated area. As explained in Section 4.3.3, the VADOFT and AT123D computer codes can be used to predict estimated loadings of contaminant to surface water as a function of the dry weight concentration of contaminants in sludge. For Antrim, the distance from near edge of the (idealized square) lagoon to the Contoocook river is approximately 100 meters, or 170 meters, from the far edge of the lagoon. To estimate loadings of contaminant to the river, the linked VADOFT-AT123D model is executed with input parameters listed in Table 4-7, combined with slightly modified input parameters for AT123D, as listed in Table 4-11. Table 4-11 differs from Table 4-9 in that the distance to the receptor well in Table 4-9 has been replaced by a lesser distance to surface water in Table 4-11, and that the x-, y- and z-source locations have been changed to reflect an even loading of contaminant throughout the entire section of aquifer beneath the impoundment. In addition, the width of the section of aquifer to be modeled has artificially constrained to that of the site. Results from execution of the linked model are presented in Table 4-12. The estimated concentration of benzene is 0.018 mg/kg (or g/m^3) throughout a cross-section of the aquifer as it intersects the Contoocook. As shown in the table, this value is multiplied by the volume of water expected to discharge to the stream from this section of aquifer (3.2 m^3/hr) to yield an estimated $5.8x10^{-5}$ kg/hr of benzene loading to the river. Since the linked model adjusts calculations within AT123D by Equation 4-2 to account for extra dilution from seepage, and since this dilution is irrelevant for the total loading of contaminant expected to reach the stream, the calculated loading is divided by the dilution factor to yield $(5.8x10^{-5})/(0.33) = 1.8x10^{-4}$ kg/hr of loading, or about 0.003% of the expected loading to the aquifer beneath the site. For convenience, this value is converted to $4.9x10^{-2}$ mg/sec, and divided by the unit concentration of benzene used as input to VADOFT to derive a ratio of loading to liquid concentration:

$$W_0/C_l = (4.9x10^{-2} \text{ mg/sec})/(1.0 \text{ mg}/l) = 4.9x10^{-2}\ l\text{/sec}.$$

TABLE 4-11

Input Parameters for AT123D Simulation of Contaminant Transport Through the Saturated Zone to Surface Water Antrim, New Hampshire

Parameter	Value	(Units)
Distance to surface water	170	(m)
Aquifer width	71	(m)
Aquifer depth	15	(m)
Begin point of x-source location	0	(m)
End point of x-source location	71	(m)
Begin point of y-source location	0	(m)
End point of y-source location	71	(m)
Begin point of z-source location	0	(m)
End point of z-source location	15	(m)
Porosity	0.43	(unitless)
Hydraulic conductivity	7.13	(m/hr)
Hydraulic gradient	0.01	(unitless)
Longitudinal dispersivity	15.3	(m)
Lateral dispersivity	5.1	(m)
Vertical dispersivity	1.0	(m)
Distribution coefficient (benzene)	0.08	(l/kg)
Distribution coefficient (lead)	234	(l/kg)
Decay constant (benzene)	2.64×10^{-4}	(hr^{-1})
Decay constant (lead)	0	(hr^{-1})
Density of soil	1.51	(kg/l)
Density of water	1.0	(kg/l)
Waste release rate (benzene)*	5.9×10^{-3}	(kg/hr)
Waste release rate (lead)*	6.4×10^{-3}	(kg/hr)
Superimposed velocity term	1.7×10^{-3}	(m/hr)

*Generated by VADOFT component of model code.

TABLE 4-12

Results from AT123D Simulation of Contaminant Transport Through the Saturated Zone to Surface Water Antrim, New Hampshire

Parameter	Value	(Units)
Darcy velocity in aquifer	3.0×10^{-3}	(m/hr)
Depth of cross section	15	(m)
Width of cross section	71	(m)
Flow through cross section	3.2	(m^3/hr)
Results from Simulation of Benzene Transport		
Steady state concentration at discharge	0.018	(g/m^3)
Time required to reach steady state	1×10^{5}	(hr)
Unadjusted loading to surface water	5.8×10^{-5}	(kg/hr)
Dilution adjustment	0.33	(unitless)
Adjusted loading to surface water	1.8×10^{-4}	(kg/hr)
W_0/C_l	4.9×10^{-2}	(l/sec)
C_l/N	2.1×10^{-2}	(kg/l)
W_0/N	1.0×10^{-3}	(kg/sec)
Results from Simulation of Lead Transport		
Steady state concentration at discharge	0.67	(g/m^3)
Time required to reach steady state	9×10^{7}	(hr)
Unadjusted loading to surface water	2.1×10^{-3}	(kg/hr)
Dilution adjustment	0.33	(unitless)
Adjusted loading to surface water	6.4×10^{-3}	(kg/hr)
W_0/C_l	1.8	(l/sec)
C_l/N	4.1×10^{-3}	(kg/l)
W_0/N	7.3×10^{-3}	(kg/sec)

This result is combined with the value estimated above for C_l/N:

$$W_0/N = (W_0/C_l)(C_l/N) = (4.9\times10^{-2}\ l/\text{sec})(0.021\ \text{kg}/l) = 1.0\times10^{-3}\ \text{kg/sec}$$

The calculations are similar for lead. Not surprisingly, the estimated loading of lead to the Contoocook River is the same as the expected loading to the aquifer, since lead is assumed to be conserved during transport. The expected concentration of lead within the artificially confined aquifer is 0.67 mg/*l*, about 2/3 the unit concentration assumed within the impoundment. This concentration can be multiplied by the expected volume of water passing through the cross-section of aquifer beneath the impoundment to derive an estimated loading of 2.1×10^{-3} kg/hr of lead to the river, which can be adjusted for the dilution factor to derive an expected loading of 6.4×10^{-3} kg/hr or 1.8 mg/sec (the same loading of contaminant calculated as the waste release rate to the aquifer). This estimated loading must be converted into a ratio of loading to dry-weight sludge concentration:

$$W_0/C_l = (1.8\ \text{mg/sec})/(1.0\ \text{mg}/l) = 1.8\ l/\text{sec}$$

and:

$$W_0/N = (1.8\ l/\text{sec})(4.1\times10^{-3}\ \text{kg}/l) = 7.3\times10^{-3}\ \text{kg/sec}$$

These results show that for every mg/kg of benzene or lead in sludge accumulating in the lagoons in Antrim, an expected 1.0×10^{-3} and 7.3×10^{-3} kg/sec of benzene or lead, respectively, are expected to be loaded into the Contoocook River. As will be discussed in Chapter 5, these ratios can be used to derive criteria for exposure pathways related to contamination of surface water.

4.7.1.2. **TULSA, OKLAHOMA** -- The Northside municipal wastewater treatment facility is designed to process 36 million gallons of wastewater per day. Sludge generated by the facility is continuously piped to three clay-lined lagoons. When a lagoon fills with sludge solids, its contents are removed for land application. Oklahoma law requires that such cleaning take place every 6 months or less; this requirement is intended to stop plants from stockpiling sludge in their lagoons. A two foot layer of sludge is usually left in the bottom of a lagoon when it is emptied, however, to prevent damage to the floor of the facility. Upon removal from the impoundment, sludge typically contains about 7-9 percent solids. Solids content in a filled lagoon range from about 6% at one foot depth to about 8% percent at the bottom of the impoundment. Sludge in the lagoons is routinely sampled for metals content, and recently was tested for the first time for organic contaminants. Groundwater monitoring wells surround the site.

The surface area of each of the three sludge lagoons at the Tulsa facility is about 2 hectares; depths of the lagoons range from about 9.5-11 feet (2.9-3.3m). For the purposes of these sample calculations, the three facilities are modeled as a single impoundment with an area of about 6 hectares. Edges of the impoundments slope inward at about a three to one slope, but for simplicity, the sample calculations described below model the impoundments as having identical bottom and surface dimensions.

Tier 2 calculations for this facility are quite similar to those for Antrim, but several of the parameter values used for the groundwater simulations are changed. Test borings beneath the site have revealed mostly silty clay to a depth of 30-40 feet (9-12 meters), but measurements of the depth to groundwater were not available. Based on data from the DRASTIC data base U.S. EPA (1985d) in GEMS, however, average depths to groundwater in the Tulsa area range from 9-15 meters. A distance of 12 meters was assumed for these calculations. Estimated aquifer thickness is based on data from the GRNDWAT data base in GEMS that report aquifer thickness ranging from 1-61 meters in the region of the Tulsa facility. A midpoint value of 30 meters is used for these sample calculations. Depths of these two layers total 42 meters. This value is used as input for the VADOFT model, as shown by Table 4-13. Characteristics of both the unsaturated and saturated zones are selected to be typical of silt. Seepage from the facility is assumed to be 0.12 l/m^2-hour, a value from the top of the range in Table 4-2 for an impoundment sealed with bentonite.

Results of the VADOFT calculations are summarized in Table 4-14. As shown in the table, the water table beneath the site is predicted to be about 1 meter higher than it would have been

TABLE 4-13

Input Parameters for VADOFT Simulation of Flow and Contaminant Transport Through the Unsaturated Zone
Tulsa, Oklahoma

Parameter	Value	(Units)
Source area	6.1×10^5	(m^2)
Distance to bottom of saturated zone	42	(m)
Thickness of Layer I	12	(m)
Thickness of Layer II	30	(m)
Input Parameters for Flow Calculations		
Flux at Top Node	0.12	(l/m^2-hr)
Head at bottom node	30	(m)
Hydraulic conductivity of Layer I	2.0×10^{-4}	(m/hr)
Hydraulic conductivity of Layer II	3.6×10^{-3}	(m/hr)
Effective porosity of Layer I	0.36	(unitless)
Effective porosity of Layer II	0.10	(unitless)
Specific Storage	0	(unitless)
Residual water saturation	0.19	(unitless)
Power index (N)	-1.0	(unitless)
Leading coefficient	0.5	(unitless)
Power index (β)	1.09	(unitless)
Power index (γ)	0.083	(unitless)
Input Parameters for Transport Calculations		
Contaminant concentration	1	(mg/l)
Flux at top node	1.2×10^{-7}	(kg/m^2-hr)
Head at bottom node	0	(m)
Longitudinal dispersivity	1.0	(unitless)
Effective porosity of Layer I	0.36	(unitless)
Effective porosity of Layer II	0.10	(unitless)
Retardation coefficient of Layer I	1.38	(unitless)
Retardation coefficient of Layer II	1.91	(unitless)
Molecular diffusion coefficient	0	(unitless)
Default Darcy velocity	0	(m/hr)
Default water saturation	1.0	(unitless)
Solute decay constant (benzene)	2.6×10^{-4}	(hr^{-1})
Solute decay constant (lead)	0	(hr^{-1})

TABLE 4-14

Results from VADOFT Execution
Tulsa, Oklahoma

Parameter	Value	(Units)
Results from Flow Simulation		
Water table elevation	2.6	(m)
Results from Simulation of Benzene Transport		
Net dispersive flux	4.8×10^{-8}	(kg/m^2-hr)
Net advective flux	7.3×10^{-8}	(kg/m^2-hr)
Cumulative mass decay	1.2×10^{-7}	(kg/m^2-hr)
Cumulative mass inflow	1.2×10^{-7}	(kg/m^2-hr)
Cumulative mass outflow	-4.0×10^{-15}	(kg/m^2-hr)
Mass flux	2.7×10^{-10}	(kg/m^2-hr)
Waste release rate	1.6×10^{-5}	(kg/hr)
Results from Simulation of Lead Transport		
Net dispersive flux	-1.2×10^{-7}	(kg/m^2-hr)
Net advective flux	1.2×10^{-7}	(kg/m^2-hr)
Cumulative mass decay	0	(kg/m^2-hr)
Cumulative mass inflow	1.2×10^{-7}	(kg/m^2-hr)
Cumulative mass outflow	-1.2×10^{-7}	(kg/m^2-hr)
Mass flux	1.2×10^{-7}	(kg/m^2-hr)
Waste release rate	7.4×10^{-3}	(kg/hr)

without the impoundment. Almost all of the benzene is removed by decay processes during its transport through the unsaturated zone, but lead concentrations are unchanged.

Table 4-15 shows inputs for the AT123D calculations. Parameter values have been selected as appropriate for an aquifer that is 30 meters thick and consists primarily of sand. The impoundment site is represented as a square of width 247 meters aligned with the direction of groundwater flow. A superimposed velocity term of 2.8×10^{-4} m/hour is estimated from the rate of seepage beneath the facility. Mass release rates of 1.6×10^{-5} kg/hour and 7.4×10^{-3} kg/hour, for benzene and lead, respectively, are taken from calculations by the VADOFT module.

Table 4-16 lists results from the AT123D model run. Because the Darcy velocity at the Tulsa site is much lower than for Antrim, a longer time interval is required to reach steady state, and there is more opportunity for contaminant decay between the site and the receptor well. As a result, expected benzene concentrations are insignificant. For every milligram per liter of concentration at the bottom of the impoundment, 2.0×10^{-9} mg/*l* are expected at the receptor well. Predicted lead concentrations, on the other hand, actually exceed the unit concentration used as input for the simulation. This physical impossibility reflects the fact that lead is not expected to decay in transport through the unsaturated zones, that the impact of seepage from the impoundment on the flow of groundwater is assumed to be significant near the site, and that the model here described relies on several conservative assumptions made necessary by reliance on a one-dimensional flow model for the unsaturated zone, and an analytical code for transport in the saturated zone.

The results described above can be used to estimate maximum dry-weight concentrations of benzene and lead to be allowed in sludge stored in the impoundments. For benzene:

$$C_{gw} / C_l = 2.0\text{x}10^{-11} \text{ (unitless)}$$

From Equation 4-1 it follows for benzene in a sludge layer with 10% solids:

$$C_l/N = 1/(37+10) = 0.021 \text{ (kg/}l\text{)}$$

TABLE 4-15

Input Parameters for AT123D Simulation of Contaminant Transport Through the Saturated Zone
Tulsa, Oklahoma

Parameter	Value	(Units)
Distance to receptor well	400	(m)
Aquifer width	infinite	(m)
Aquifer depth	30	(m)
Begin point of x-source location	0	(m)
End point of x-source location	247	(m)
Begin point of y-source location	-123.5	(m)
End point of y-source location	123.5	(m)
Porosity	0.10	(unitless)
Hydraulic conductivity	0.86	(m/hr)
Hydraulic gradient	0.01	(unitless)
Longitudinal dispersivity	15.3	(m)
Lateral dispersivity	5.1	(m)
Vertical dispersivity	1.0	(m)
Distribution coefficient (benzene)	0.08	(l/kg)
Distribution coefficient (lead)	234	(l/kg)
Decay constant (benzene)	2.64×10^{-4}	(hr^{-1})
Decay constant (lead)	0	(hr^{-1})
Density of soil	2.38	(kg/l)
Density of water	1.0	(kg/l)
Waste release rate (benzene)*	1.9×10^{-4}	(kg/hr)
Waste release rate (lead)*	9.2×10^{-3}	(kg/hr)
Superimposed velocity term	2.8×10^{-4}	(m/hr)

*Generated by VADOFT component of model code.

TABLE 4-16

Results from AT123D Simulation of Contaminant Transport Through the Saturated Zone
Tulsa, Oklahoma

Parameter	Value	(Units)
Results from Simulation of Benzene Transport		
Retardation factor	2.90	(unitless)
Retarded Darcy velocity	1.3×10^{-3}	(m/hr)
Retarded longitudinal dispersion coefficient	0.02	(m^2/hr)
Retarded lateral dispersion coefficient	6.75×10^{-3}	(m^2/hr)
Retarded vertical dispersion coefficient	1.3×10^{-3}	(m^2/hr)
Steady state concentration at receptor well	2.0×10^{-9}	(mg/*l*)
Results from Simulation of Lead Transport		
Retardation factor	5570	(unitless)
Retarded Darcy velocity	6.9×10^{-7}	(m/hr)
Retarded longitudinal dispersion coefficient	1.05×10^{-5}	(m^2/hr)
Retarded lateral dispersion coefficient	3.5×10^{-6}	(m^2/hr)
Retarded vertical dispersion coefficient	6.9×10^{-7}	(m^2/hr)
Steady state concentration at receptor well	1.1	(mg/*l*)

Then from Equation 4-3:

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = (2.0\times10^{-11})(0.021) = 4.3\times10^{-13}\ (kg/l)$$

and from Equation 4-4:

$$N_{max} = RWC\ /\ SRR_{GW} = (1.2\times10^{-3})/(4.3\times10^{-13}) = 2.8\times10^{9}\ (mg/kg)$$

From these calculations it could be concluded that no possible concentration of benzene in the impoundment would be expected to result in groundwater concentrations that exceed the RWC. Similar calculations can be performed for lead, for which the concentration in well water is predicted to be the same as the dissolved concentration at the floor of the lagoon:

$$C_{gw}/C_l = 1.0\ \ (unitless)$$

$$C_l/N = 4.1\times10^{-3}\ kg/l$$

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = (1)(4.1\times10^{-3}) = 4.1\times10^{-3}\ kg/l$$

$$N_{max} = RWC\ /\ SRR_{GW} = (0.005)/(4.1\times10^{-3}) = 1.2\ mg/kg$$

The resulting criteria would not be limiting for benzene, but would be quite restrictive for lead. If actual concentrations in the sludge exceed these criteria, then long-term storage of sludge in the lagoons would not be allowed.

Results from VADOFT and AT123D can also be used to estimate expected loadings of lead and benzene to Bird Creek, which is located about 50 meters from the lagoons at the Northside facility (or about 300 meters from the far edge of an idealized square impoundment). If it is assumed that groundwater flows toward the creek, then it can be further assumed that the loading of lead from the aquifer to the creek will be the same as the estimated loading of lead to the aquifer (7.4 kg/hr or 2.0 mg/sec) so that W_0/C_l = (2.0 mg/sec)/(1 mg/*l*) = 2.0 *l*/sec. From this result it follows that:

$$W_0/N = (W_0/C_l)(C_l/N) = (2.0\ l/\text{sec})(4.1\text{x}10^{-3}\ \text{kg}/l) = 8.3\text{x}10^{-3}\ \text{kg/sec.}$$

For benzene, the linked VADOFT-AT123D model is used to predict the concentration of contaminant within an artificially confined aquifer with width corresponding to that of the site. Inputs for the VADOFT model as listed in Table 4-16, combined with inputs for the AT123D model, as listed in Table 4-17, are supplied to the model to yield results listed in Table 4-18. As before, AT123D is used to estimate expected loadings to the creek, based on a scenario in which the contaminant loading is uniformly distributed throughout the aquifer beneath the site, and in which transverse dispersion is constrained by confining the aquifer to a width corresponding to that of the site. Results from AT123D indicate an expected steady-state concentration of $1.5\text{x}10^{-7}$ g/m^3 (or mg/*l*) at a distance of 50 meters from the edge of the site. Based on a hydraulic conductivity of 0.86 m/hr and a hydraulic gradient of 0.1, the Darcy velocity of water within the aquifer is $3.6\text{x}10^{-5}$ m/hr, suggesting 0.27 m^3 of flow per hour through a cross-section of 30 x 247 meters. The estimated concentration is multiplied by the expected volume of flow to yield $3.9\text{x}10^{-11}$ kg/hour of loading to the creek. This estimate is adjusted for a dilution factor of 0.035, converted to convenient units and divided by the unit concentration used for the calculations to yield

$$W_0/C_l = (3.8\text{x}10^{-7}\ \text{mg/sec})/(1\ \text{mg}/l) = 3.1\text{x}10^{-7}\ l/\text{sec}$$

and

$$W_0/N = (W_0/C_l)(C_l/N) = (3.1\text{x}10^{-7}\ l/\text{sec})(2.1\text{x}10^{-2}\ \text{kg/l}) = 6.6\text{x}10^{-9}\ \text{kg/sec}$$

This ratio will be used in Chapter 5 to derive criteria for surface water pathways.

TABLE 4-17

Input Parameters for AT123D Simulation of Contaminant Transport Through the Saturated Zone to Surface Water Tulsa, Oklahoma

Parameter	Value	(Units)
Distance to surface water	300	(m)
Aquifer width	247	(m)
Aquifer depth	30	(m)
Begin point of x-source location	0	(m)
End point of x-source location	247	(m)
Begin point of y-source location	0	(m)
End point of y-source location	247	(m)
Begin point of z-source location	0	(m)
End point of z-source location	30	(m)
Porosity	0.10	(unitless)
Hydraulic conductivity	0.86	(m/hr)
Hydraulic gradient	0.01	(unitless)
Longitudinal dispersivity	15.3	(m)
Lateral dispersivity	5.1	(m)
Vertical dispersivity	1.0	(m)
Distribution coefficient (benzene)	0.08	(l/kg)
Distribution coefficient (lead)	234	(l/kg)
Decay constant (benzene)	2.64×10^{-4}	(hr^{-1})
Decay constant (lead)	0	(hr^{-1})
Density of soil	2.38	(kg/l)
Density of water	1.0	(kg/l)
Waste release rate (benzene)*	1.9×10^{-4}	(kg/hr)
Waste release rate (lead)*	9.2×10^{-3}	(kg/hr)
Superimposed velocity term	3.5×10^{-4}	(m/hr)

*Generated by VADOFT component of model code.

TABLE 4-18

Results from AT123D Simulation of Contaminant Transport Through the Saturated Zone to Surface Water Tulsa, Oklahoma

Parameter	Value	(Units)
Darcy velocity in aquifer	$3.6x10^{-5}$	(m/hr)
Depth of cross section	30	(m)
Width of cross section	247	(m)
Flow through cross section	0.27	(m^3/hr)
Results from Simulation of Benzene Transport		
Steady state concentration at discharge	$1.5x10^{-7}$	(g/m^3)
Time required to reach steady state	$4x10^6$	(hr)
Unadjusted loading to surface water	$3.9x10^{-11}$	(kg/hr)
Dilution adjustment	0.035	(unitless)
Adjusted loading to surface water	$1.1x10^{-9}$	(kg/hr)
W_0/C_l	$3.1x10^{-7}$	(*l*/sec)
C_l/N	0.021	(kg/*l*)
W_0/N	$6.6x10^{-9}$	(kg/sec)
Results from Simulation of Lead Transport		
Steady state concentration at discharge	0.97	(g/m^3)
Time required to reach steady state	$8x10^9$	(hr)
Unadjusted loading to surface water	$2.6x10^{-4}$	(kg/hr)
Dilution adjustment	0.035	(unitless)
Adjusted loading to surface water	$7.3x10^{-3}$	(kg/hr)
W_0/C_l	2.0	(*l*/sec)
C_l/N	$4.1x10^{-3}$	(kg/*l*)
W_0/N	$8.3x10^{-3}$	(kg/sec)

4.7.1.3. PORTLAND, OREGON -- The Columbia Boulevard wastewater treatment plant in Portland has been in operation since the 1940's and was updated in the 1970's to include secondary treatment. In the past, sludge was discharged directly to an on-site sludge lagoon; some of the sludge from the lagoon was then pumped out and processed in a composting facility. Now, almost all of the plant's sludge goes directly to the in-vessel composting facility. The impoundment is used only in emergency situations, upon failure of the composting process. Recently, for example, problems with the composting process resulted in the pumping of 260,000 gallons of sludge to the lagoon.

The lagoon in Portland is estimated to cover about 32 acres (13 hectares) to a depth of about 16 feet (5 meters), and is currently estimated to contain about 80,000 dry tons of sludge. Plans for future operation of the facility are not certain; the city hopes to exhume all of the sludge eventually for composting, but the in-vessel composting system can handle only the current volume of sludge generated. Sludge quality in the impoundment varies by location, with older deposits typically containing higher concentrations of metals and PCBs. The impoundment is located about 50 meters from the Columbia Slough, which empties into the Columbia River.

Soil borings for the Portland site were not available. These sample calculations therefore rely on data from the DRASTIC data base accessed through GEMS (U.S. EPA, 1988c). Based on hydrogeological data for the county containing the lagoon, these sample calculations assume that the vadose zone is composed of sand to a depth of 15 meters, followed by a saturated zone of sand with a thickness of 15 meters. These two layers sum to a total distance of 30 meters from the floor of the lagoon to the bottom of the saturated zone.

Table 4-19 contains values for input parameters used to simulate flow and contaminant transport beneath the Portland facility. Table 4-20 lists results from the VADOFT simulation, and shows that the water table beneath the site is expected to show an increase of 0.6 meters in elevation as a result of seepage from the impoundment. Benzene concentrations decrease by about 13% as a result of travel through the unsaturated zone.

Table 4-21 lists inputs for estimating contaminant transport through the saturated zone. As for Antrim, the saturated zone is assumed to consist of a layer of sand 15 meters thick and infinitely wide. Waste release rates are taken from VADOFT, and the distance to the well is set to equal the

TABLE 4-19

Input Parameters for VADOFT Simulation of Flow and Contaminant Transport Through the Unsaturated Zone
Portland, Oregon

Parameter	Value	(Units)
Source area	1.30×10^5	(m^2)
Distance to bottom of saturated zone	30	(m)
Input Parameters for Flow Calculations		
Flux at Top Node	1.3	(l/m^2-hr)
Head at bottom node	15	(m)
Hydraulic conductivity	0.30	(m/hr)
Effective porosity	0.43	(unitless)
Specific Storage	0	(unitless)
Residual water saturation	0.105	(unitless)
Power index (N)	-1.0	(unitless)
Leading coefficient	14.5	(unitless)
Power index (β)	2.68	(unitless)
Power index (γ)	0.62	(unitless)
Input Parameters for Transport Calculations		
Contaminant concentration	1	(mg/l)
Flux at top node	1.3×10^{-6}	(kg/m^2-hr)
Head at bottom node	0	(m)
Longitudinal dispersivity	1.0	(unitless)
Effective porosity	0.43	(unitless)
Retardation coefficient	1.28	(unitless)
Molecular diffusion coefficient	0	(unitless)
Default Darcy velocity	0	(m/hr)
Default water saturation	1.0	(unitless)
Solute decay constant (benzene)	2.6×10^{-4}	(hr^{-1})
Solute decay constant (lead)	0	(hr^{-1})

TABLE 4-20

Results from VADOFT Execution
Portland, Oregon

Parameter	Value	(Units)
Results from Flow Simulation		
Water table elevation	0.6	(m)
Results from Simulation of Benzene Transport		
Net dispersive flux	$-1.1\text{x}10^{-7}$	(kg/m^2-hr)
Net advective flux	$1.2\text{x}10^{-6}$	(kg/m^2-hr)
Cumulative mass decay	$1.1\text{x}10^{-6}$	(kg/m^2-hr)
Cumulative mass inflow	$1.3\text{x}10^{-6}$	(kg/m^2-hr)
Cumulative mass outflow	$-1.6\text{x}10^{-7}$	(kg/m^2-hr)
Mass flux	$6.9\text{x}10^{-7}$	(kg/m^2-hr)
Waste release rate	$9.0\text{x}10^{-2}$	(kg/hr)
Results from Simulation of Lead Transport		
Net dispersive flux	$-1.3\text{x}10^{-6}$	(kg/m^2-hr)
Net advective flux	$1.3\text{x}10^{-6}$	(kg/m^2-hr)
Cumulative mass decay	0	(kg/m^2-hr)
Cumulative mass inflow	$1.3\text{x}10^{-6}$	(kg/m^2-hr)
Cumulative mass outflow	$-1.3\text{x}10^{-6}$	(kg/m^2-hr)
Mass flux	$1.3\text{x}10^{-6}$	(kg/m^2-hr)
Waste release rate	$1.7\text{x}10^{-1}$	(kg/hr)

TABLE 4-21

Input Parameters for AT123D Simulation of Contaminant Transport Through the Saturated Zone Portland, Oregon

Parameter	Value	(Units)
Distance to receptor well	510	(m)
Aquifer width	infinite	(m)
Aquifer depth	15	(m)
Begin point of x-source location	0	(m)
End point of x-source location	361	(m)
Begin point of y-source location	-180	(m)
End point of y-source location	180	(m)
Porosity	0.43	(unitless)
Hydraulic conductivity	0.30	(m/hr)
Hydraulic gradient	0.01	(unitless)
Longitudinal dispersivity	15.3	(m)
Lateral dispersivity	5.1	(m)
Vertical dispersivity	1.0	(m)
Distribution coefficient (benzene)	0.08	(l/kg)
Distribution coefficient (lead)	234	(l/kg)
Decay constant (benzene)	2.64×10^{-4}	(hr^{-1})
Decay constant (lead)	0	(hr^{-1})
Density of soil	1.51	(kg/l)
Density of water	1.0	(kg/l)
Waste release rate (benzene)*	8.9×10^{-5}	(kg/hr)
Waste release rate (lead)*	1.65×10^{-4}	(kg/hr)
Superimposed velocity term	8.6×10^{-3}	(m/hr)

*Generated by the VADOFT component of the model code.

distance to the property boundary. With these input values, AT123D produces the results listed in Table 4-22. Concentrations of benzene are expected to decrease by about 98% between the impoundment and the receptor well, but concentrations of lead decrease only 9%. These results can be used to derive maximum allowable concentrations of lead and benzene in sludge deposited in the Portland facility. For benzene,

$$C_{gw}/C_l = 0.019 \text{ (unitless)}$$
$$C_l/N = 0.021 \text{ (kg/}l\text{)}$$

Then from Equation 4-3:

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = (0.019)(0.021) = 4.0 \times 10^{-4}$$

and from Equation 4-4:

$$N_{max} = RWC \,/\, SRR_{GW} = (1.2\text{x}10^{-3})/(4.0\text{x}10^{-4}) = 3.0 \text{ mg/kg}$$

Similar calculations can be performed for lead:

$$C_{gw}/C_l = 0.91$$

$$C_l/N = 4.1\text{x}10^{-3}$$

$$SRR_{GW} = (C_{gw}/C_l)(C_l/N) = (0.91)(4.1\text{x}10^{-3}) = 3.7\text{x}10^{-3}$$

$$N_{max} = RWC \,/\, SRR_{GW} = (0.005)/(3.7\text{x}10^{-3}) = 1.3 \text{ mg/kg}$$

TABLE 4-22

Results from AT123D Simulation of Contaminant Transport Through the Saturated Zone Portland, Oregon

Parameter	Value	(Units)
Results from Simulation of Benzene Transport		
Retardation factor	1.28	(unitless)
Retarded Darcy velocity	0.02	(m/hr)
Retarded longitudinal dispersion coefficient	0.32	(m^2/hr)
Retarded lateral dispersion coefficient	0.11	(m^2/hr)
Retarded vertical dispersion coefficient	0.02	(m^2/hr)
Steady state concentration at receptor well	19	(μg/l)
Results from Simulation of Lead Transport		
Retardation factor	823	(unitless)
Retarded Darcy velocity	3.3×10^{-5}	(m/hr)
Retarded longitudinal dispersion coefficient	5.0×10^{-4}	(m^2/hr)
Retarded lateral dispersion coefficient	1.7×10^{-4}	(m^2/hr)
Retarded vertical dispersion coefficient	3.3×10^{-5}	(m^2/hr)
Steady state concentration at receptor well	907	((μg/l)

If actual concentrations in the sludge exceed these criteria, then long-term (or permanent) storage of sludge in the lagoons would not be allowed.

Results from VADOFT calculations are next used to estimate expected loadings of benzene and lead to the Columbia Slough, located about 50 meters from the lagoon. It is assumed that the loading of lead to surface water is the same as the estimated loading to groundwater, or 0.16 kg/hr. This value is converted to mg/sec and divided by the unit concentration in the impoundment (1 mg/*l*) to derive W_0/C_l = 46 *l*/sec, which is then combined with C_l/N to yield:

$$W_0/N = (W_0/C_l)(C_l/N) = (46)(4.1\mathrm{x}10^{-3}) = 0.19 \text{ kg/sec}$$

For benzene, methods identical to those described in sections 4.6.1 and 4.6.2 are used to predict expected loadings to the Columbia Slough, based on input parameters listed in Table 4-23. As shown in Table 4-24, the model predicts a steady-state concentration of 0.055 mg/*l* (or g/m^3). This result is multiplied by the estimated flow of water through a cross section of the aquifer, and adjusted to compensate for dilution calculations in the linked model:

$$W_0 = (0.055 \text{ mg}/l)(3.0\mathrm{x}10^{-3})(15)(361)/(0.089) = 2.8 \text{ mg/sec}$$

so:

$$W_0/C_l = (2.8 \text{ mg/sec})/(1 \text{ mg}/l) = 2.8\ l\text{/sec}$$

This ratio is combined with an estimate of C_l/N to compare loading to the dry-weight concentration of benzene in sludge received by the lagoon:

$$W_0/N = (W_0/C_l)(C_l/N) = (2.8\ l\text{/sec})(0.021 \text{ kg}/l) = 0.059 \text{ kg/sec}$$

These values will be used to derive criteria in Chapter 5.

TABLE 4-23

Input Parameters for AT123D Simulation of Contaminant Transport Through the Saturated Zone to Surface Water Portland, Oregon

Parameter	Value	(Units)
Distance to surface water	410	(m)
Aquifer width	361	(m)
Aquifer depth	15	(m)
Begin point of x-source location	0	(m)
End point of x-source location	361	(m)
Begin point of y-source location	0	(m)
End point of y-source location	361	(m)
Begin point of z-source location	0	(m)
End point of z-source location	15	(m)
Porosity	0.43	(unitless)
Hydraulic conductivity	0.30	(m/hr)
Hydraulic gradient	0.01	(unitless)
Longitudinal dispersivity	15.3	(m)
Lateral dispersivity	5.1	(m)
Vertical dispersivity	1.0	(m)
Distribution coefficient (benzene)	0.08	(l/kg)
Distribution coefficient (lead)	234	(l/kg)
Decay constant (benzene)	2.64×10^{-4}	(hr^{-1})
Decay constant (lead)	0	(hr^{-1})
Density of soil	1.51	(kg/l)
Density of water	1.0	(kg/l)
Waste release rate (benzene)*	8.9×10^{-5}	(kg/hr)
Waste release rate (lead)*	1.6×10^{-4}	(kg/hr)
Superimposed velocity term	8.6×10^{-3}	(m/hr)

*Generated by the VADOFT component of the model code.

TABLE 4-24

Results from AT123D Simulation of Contaminant Transport Thorough the Saturated Zone to Surface Water Portland, Oregon

Parameter	Value	(Units)
Darcy velocity in aquifer	0.003	(m/hr)
Depth of cross section	15	(m)
Width of cross section	361	(m)
Flow through cross section	16	(m^3/hr)
Results from Simulation of Benzene Transport		
Steady state concentration at discharge	0.055	(g/m^3)
Time required to reach steady state	1×10^5	(hr)
Unadjusted loading to surface water	8.9×10^{-4}	(kg/hr)
Dilution adjustment	0.089	(unitless)
Adjusted loading to surface water	1×10^{-2}	(kg/hr)
W_0/C_l	2.8	(*l*/sec)
C_l/N	0.021	(kg/*l*)
W_0/N	0.059	(kg/sec)
Results from Simulation of Lead Transport		
Steady state concentration at discharge	0.91	(g/m^3)
Time required to reach steady state	8×10^7	(hr)
Unadjusted loading to surface water	0.015	(kg/hr)
Dilution adjustment	0.089	(unitless)
Adjusted loading to surface water	0.16	(kg/hr)
W_0/C_l	46	(*l*/sec)
C_l/N	4.1×10^{-3}	(kg/*l*)
W_0/N	0.19	(kg/sec)

4.7.2. Analysis of Exposure for the Most Exposed Populations. As discussed in Chapter 3, an analysis of exposure to most exposed populations (MEP) can be useful for understanding the human health risks associated with a particular sludge disposal option or exposure pathway. For the groundwater pathway, potential exposure is unlikely to extend beyond a small geographical area surrounding the facility, and is likely to be confined to a small population. In cases where public water supply systems withdraw water from the contaminated aquifer, the size of the potentially exposed population will be larger, but variation in individual exposure will be determined primarily by variation in individual water consumption behavior. Methodological difficulties in quantifying individual variation in water ingestion rates (I_w) complicate efforts to determine the distribution of potential exposure among those ingesting water from a contaminated aquifer. As a result of these constraints, no analysis of most exposed populations will be presented for the groundwater pathway.

5. DERIVATION OF CRITERIA FOR THE SURFACE WATER PATHWAY

This chapter describes and demonstrates a method for modeling the long-term fate and transport in surface water of selected contaminants from a surface disposal site for wastewater sludge.

5.1 OVERVIEW OF THE METHOD

Municipal sewage treatment plants are usually found near natural surface water courses like rivers, lakes or estuaries. For simplicity of presentation, it is assumed that the site is located next to a river, although the methodology is equally applicable to other types of water bodies. The sludge disposal facility is assumed to be an unlined impoundment enclosed by an engineered embankment, probably constructed from on-site material. It is assumed that the site is properly graded so that rainfall runoff from upstream areas is diverted around and not into the sludge disposal impoundment. Therefore, since runoff from the site is unlikely to carry contaminants to nearby surface water, the methodology considers only one pathway of surface water contamination: contaminants from the impoundment are transported by seepage into an aquifer beneath the site. They are then transported within the aquifer and released to the nearby surface water body.

To set criteria for sludge based on protection of surface water, three pathways must be considered: exposure of aquatic wildlife, human exposure through drinking water and human exposure through fish consumption. At sites where consumption of groundwater is also a concern, criteria are developed for the protection of drinking water uses of groundwater, based on methods outlined in Chapter 4. If the point of compliance for the groundwater pathway is of no greater distance than the nearest distance to surface water, then sludge criteria derived to maintain groundwater at healthful levels will also serve as criteria for protection of drinking water uses of surface water, since contaminant concentrations can only decrease upon further dilution from mixing with the stream. At sites where groundwater is not considered usable for current or future needs (i.e., at sites where no criteria need be developed based on the groundwater pathway) and at sites where surface water is closer than all possible well locations, surface water modeling can be used to derive sludge criteria for the protection of drinking water. For some contaminants, potential risks to aquatic life or to humans through fish consumption may occur at lower levels of surface water concentrations than those considered acceptable on the basis of drinking water criteria alone. For these, estimated exposure through the surface water pathway must be considered even if estimated concentrations in the groundwater are considered acceptable for drinking by humans.

Since the river and the sludge impoundment are assumed to be in close proximity, the river is taken as the primary surface water pathway for transporting sewage sludge contaminants away from the disposal site. Sludge contaminant loading to the river can be represented as either an instantaneous pulse or a continuous step input. For example, an instantaneous pulse could include sludge released during an embankment failure, impoundment overflow, pipeline rupture or other catastrophic accident at the disposal site. Instantaneous pulses are useful when investigating acute impacts. In contrast, continuous inputs describe loadings of extended duration, more likely to be appropriate for the loading of contaminants to a stream from groundwater inflow. Continuous inputs are useful in studies of chronic effects.

The surface water methodology assumes that contaminants migrate in seepage from an unlined impoundment to the groundwater, which in turn discharges to the river as a continuous distributed source. Consequently, the methodology considers continuous rather than instantaneous sludge loadings and is concerned with long-term chronic, rather than short-term acute effects. This mass loading of contaminant from groundwater is estimated from the steady-state response of a groundwater contaminant transport model, as described in Chapter 4.

If potential impacts from shorter-term exposure to higher, transient concentrations of contaminant are also to be considered, the methodology described in this chapter can be modified slightly to estimate potential acute exposure based on conditions of low stream flow. For example, values of the lowest 7-day flow expected to occur every 10 years (7Q10) are available for many streams from district offices of the U.S. Geological Survey. It is assumed that short-term peak loadings of contaminant from the aquifer to the stream, if they occur, are unlikely to coincide with periods of low stream flow; estimation of potential acute exposure is thus based on average, steady state loadings to the stream.

The proposed methodology for deriving criteria based on exposure through surface water pathways relies on a two-tiered approach. As in Chapter 4, Tier 1 involves the derivation of national, numerical criteria based on a "generic" scenario assumed to represent a reasonable worst case. For example, such a scenario might involve the discharge of groundwater into a lake or stream with relatively low flow; and the withdrawal of surface water for human consumption, the consumption of freshwater fish, and the exposure of aquatic wildlife in an area close to the zone of recharge. Tier 2 involves site-specific modeling of potential exposure. Those facilities that fail to satisfy the Tier 1 criteria can elect to perform a site-specific Tier 2 analysis.

Both tiers rely on a relatively simple yet flexible analytical approach in which the river is modeled as a cascade of well-mixed cells (Stefan and Demetracopoulos, 1981). Output from each upstream cell is used as input to the next downstream cell. Computations proceed downstream along the cascading cells from the point of contaminant entry to the location of the most exposed individual (MEI) or most exposed unit (MEU). This "cascading cells" model accounts for degradation processes and tributary inflows. Input parameters can be obtained from site-specific reports, maps and other existing sources, or from the literature. In addition, the cascading cells model can easily accommodate "reverse application": if the maximum allowable contaminant concentration at the MEI location is specified, the cascading cells model can be used to determine the corresponding allowable mass loading from the contaminated aquifer to the stream. This in turn can be used to calculate the maximum dry weight concentration for the contaminant in the surface impoundment.

Computer models are available for more sophisticated analyses of the fate and transport of chemical contaminants in surface water bodies (U.S. EPA, 1988b). For example, the Water Quality Analysis Simulation Program, or WASP4 model (U.S. EPA, 1988a), employs a network of segments connected by channel links to simulate the transport of contaminants in dissolved and solid phases in rivers, ponds, lakes, reservoirs and estuaries. Model segments include up to four zones (epilimnion, hypolimnion, upper benthic and lower benthic), which can be used to divide the water column and the sediment bed laterally, vertically and longitudinally. WASP4 can be employed at various levels of complexity involving increasing sophistication in solids behavior, equilibrium reactions and kinetic reactions. Transfer processes include sorption, ionization and volatilization; transformation processes include biodegradation, hydrolysis, photolysis and chemical oxidation. Sorption and ionization are treated as equilibrium reactions described by a single partition coefficient. All other processes are described by kinetic rate equations, each characterized by constant half-life or rate constant.

A similar model, the Exposure Analysis Modeling System (EXAMSII), is available within GEMS (U.S. EPA, 1988c). Chemical fate processes considered by the EXAMSII model include volatilization, hydrolysis, oxidation, reduction, photo-oxygenation, biolysis and photolysis. Like WASP4, this model simulates the transport of contaminants between multiple compartments in the stream, and considers interactions between sediment and the water column. For many locations, values for input parameters required by the model can be drawn from data bases accessible within GEMS.

Effective use of the capabilities of WASP4, EXAMSII, or similar models requires numerous site-specific input parameter values that may be difficult or expensive to obtain for certain sites, however. In addition, discharge of some species of contaminant from the aquifer to the stream may occur years or even decades after the contaminants first migrate into the aquifer. Since exact locations of future withdrawal of drinking water, or of contact between surface water and aquatic species cannot be known with certainty, the extra precision gained by using WASP4, EXAMSII or equivalent models is not necessarily advantageous for deriving criteria for sludge disposal facilities. Without consideration of additional contaminant loss processes associated with the stream floor, adsorption and accumulation of metals and high molecular weight materials at the bottom of the stream, WASP4 reduces to the equations described below. Those equations provide simpler, but more conservative, estimates of contaminant concentrations in surface water. If sufficient data are available, and more accurate (less conservative) calculations are desired, the additional capabilities of a more complete mathematical model should be utilized.

5.2. ASSUMPTIONS

A number of hydraulic and water quality assumptions are required for the cascading cells model; these are summarized in Table 5-1. The primary motivation for these assumptions is to formulate a surface water transport model that provides a reasonable representation of the receiving water body and yet can be solved analytically.

The hydraulic geometry of each cell is assumed to be fixed; that is, the length, width, depth, slope and volumetric flow of each cell in the cascade are constant, although these parameters will usually vary among cells. For computing yearly average human exposure, or for an evaluation of chronic exposure for aquatic species, the discharge through any given cell is assumed to be steady and uniform at a rate equal to the mean annual discharge of the river at the point in question. Comparisons with criteria for short-term or acute exposure should be based on seven-day, ten-year flow rates (7Q10). To simplify the calculations, it is conservatively assumed that the suspended sediment distribution is constant and that the sediment bed is stationary, although neither of these two conditions is likely to be observed in actual streams. Consideration of storm conditions is not included in the analysis of chronic exposure since transient events are assumed to have a negligible effect over the long term. Dilution effects of tributary inflows are accommodated by defining cell boundaries at each confluence along the study reach.

Contaminant mass loading from the sludge disposal site to the river is assumed to occur at a constant rate. Neither upstream nor instream sources of contaminant are considered. To reduce input requirements and simplify the calculations, it is assumed that there is no net loss or gain of contaminant through interactions with the sediment bed. The input mass is assumed to mix instantaneously and completely (i.e., vertically, laterally and longitudinally) throughout each cell. First-order kinetics with a constant decay parameter are assumed to provide an adequate description of organic chemical transformation and degradation mechanisms. It is assumed that heavy metals neither decay

TABLE 5-1

Assumptions for Methodology to Analyze Surface Water Pathways

Functional Area	Assumption	Ramifications
Contaminant Transport in Surface Water	Loading of contaminant to the stream is steady-state.	Steady-state loading may not be reached for many decades for some contaminants at some sites. Will over-predict loading in period before steady-state is reached.
	Upstream/instream sources of contaminant are not considered.	Only incremental concentration is considered.
	Hydraulic geometry of each cell is fixed.	Unknown.
	For estimating chronic exposure, it is assumed that mean annual steady state conditions prevail for all hydraulic, sediment and water quality input parameters.	Does not account for short-term fluctuations or seasonal variations in contaminant concentrations that occur during the year.
	For estimating acute exposure, it is assumed that flow is defined by the lowest expected 7-day flow per 10 years (7Q10), but loading to stream is steady-state.	Acute exposure could be higher if low stream flow coincides with period of high discharge from aquifer.
	There is instantaneous and complete mixing of the contaminant in cross section.	Underpredicts concentrations, especially near the loading source.
	The sediment bed is stationary and there is no contaminant loss or gain due to sediment and/or benthic interaction.	If the sediment bed is a sink, this assumption overpredicts contaminant concentrations; if the sediment bed is a source, it underpredicts concentrations.
	Partitioning of contaminants between dissolved and adsorbed phases is instantaneous, and can be described by a linear isotherm characterized by a single partition coefficient for each cell.	May underpredict dissolved concentrations if equilibrium is not attained.

TABLE 5-1 (cont.)

Functional Area	Assumption	Ramifications
	Degradation and transformation of organic contaminants can be described by first order kinetics.	Unknown.
	Heavy metals neither decay nor volatilize.	This assumption overpredicts heavy metal concentrations to the extent that transformations are important.
Drinking Water Pathway of Exposure	MEI ingests water at concentrations predicted for specified location downstream of the zone of aquifer discharge.	May over- or under-predict actual exposure if future location of MEI differs from specified location.
	Exposure can be predicted based on dissolved concentration of contaminant.	Underpredicts exposure to the extent that suspended solids are not removed during water treatment prior to ingestion.
Fish Consumption Pathway of Exposure	MEI ingests fish with contaminant concentrations predicted on the basis of total contaminant concentration (dissolved and adsorbed) in the first modeled cell below the zone of aquifer discharge.	Likely to overpredict concentration of contaminant in fish tissue if dissolved concentration is better predictor of bioconcentration under field conditions. Use of estimated concentrations from first cell may overpredict concentrations in fish tissue if fish inhabit a broader range within the stream.

nor volatilize. Adsorption/desorption processes are assumed to attain instantaneous equilibrium according to a linear isotherm characterized with a single partition coefficient.

5.3. CALCULATIONS

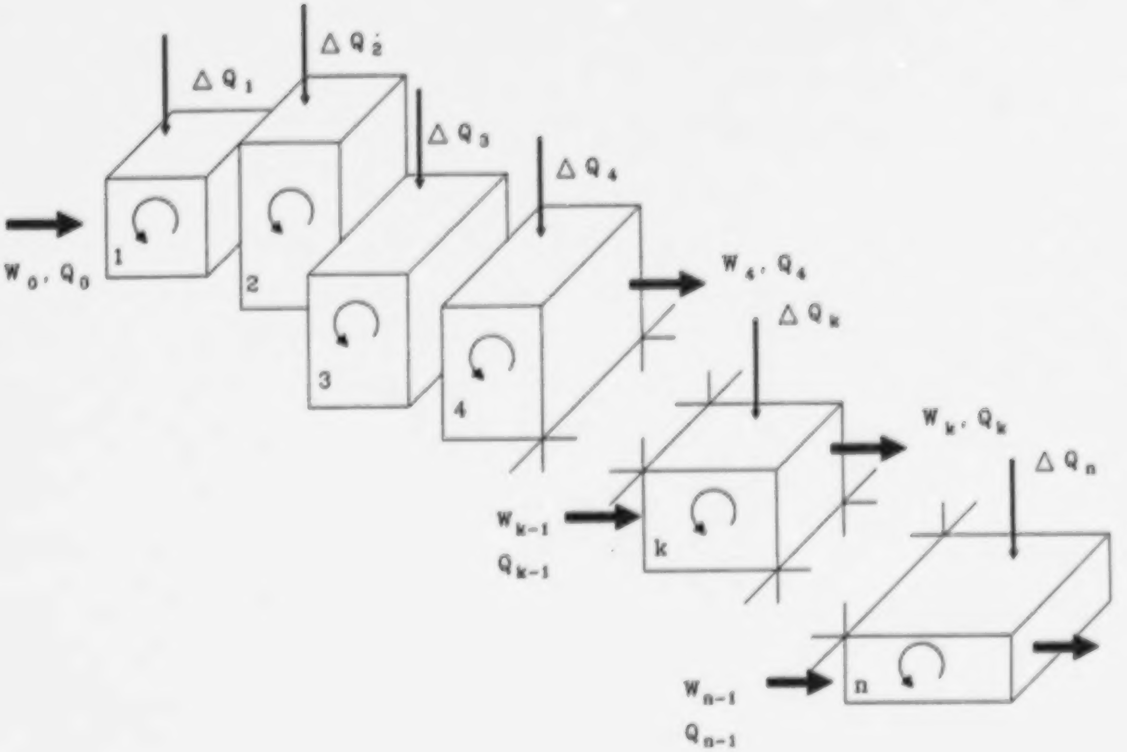
Derivation of criteria through either Tier 1 or Tier 2 involves four steps for each contaminant:

1. derive reference water concentrations (RWC) for the surface water body;
2. determine the loading of contaminant from the aquifer to the stream, as a function of the dry-weight concentrations of contaminant in sludge received by the lagoon;
3. determine downstream concentrations of contaminant in the stream as a function of loadings from the aquifer and
4. link the results of steps (1) through (3) to determine the maximum allowable concentration of the contaminant in sludge received by the surface disposal facility.

The first of these steps is the derivation of reference water concentrations. These describe the levels of contaminant concentrations in a surface water body that are not expected to cause adverse effects on human health or aquatic wildlife. Steps for deriving the RWC for surface water will be discussed in Section 5.5.

Next, the model described in Chapter 4 can be used to determine expected loadings of each contaminant to the stream, based on dry-weight concentrations of contaminant in sludge received by or accumulating in the surface impoundment. As explained in Chapter 4, results from VADOFT and AT123D simulations are linear with respect to contaminant concentrations in the sludge, so that a ratio (W_0/N) can be derived for each contaminant that describes the relationship between loading to the stream and the concentration of the contaminant in sludge.

Once a downstream point of compliance has been selected for the calculations, the cascading cells model can estimate the downstream concentrations of a contaminant as a function of its loading from the aquifer to the stream. Within the model, the study reach is depicted as a series of well-mixed cascading cells, as shown in Figure 5-1. The contaminant outflow from an upstream cell represents the contaminant inflow to the next downstream cell. From the principle of mass conservation, the steady-state concentration



ΔQ1
ΔQ2
ΔQ3
ΔQ4
W0, Q0
1
2
3
4
W4, Q4
ΔQk
Wk, Qk
ΔQn
Wk-1
Qk-1
k
Wn-1
Qn-1
n

FIGURE 5-1

Definition Sketch for Cascading Cells Model in the nth cell of the cascade is:

$$C_n = \frac{W_{n-1}}{Q_{fn} + A_n K_{totn}} \tag{5-1}$$

where:

C_n = concentration of contaminant in nth cell of cascade (g/m^3)
W_{n-1} = contaminant mass inflow from upstream cell (g/sec)
Q_{fn} = river discharge in nth cell of cascade (m^3/sec)
A_n = top surface area of nth cell of cascade (m^2)
K_{totn} = net contaminant loss rate of nth cell of cascade (m/sec)

Equation 5-1 shows that the concentration of contaminant C_n, in the downstream cell is directly proportional to W_{n-1}, the contaminant mass loading received from the cell immediately upstream. Further, cell concentrations are inversely proportional to the sum of two processes in the cell, dilution and degradation, as represented by Q_{fn} and A_nK_{totn}, respectively. For convenience, it is assumed that the loading of contaminant into the first cell, W_0, originates entirely from the aquifer; background contaminant levels from upstream sources are ignored. Within this context, C_n is best interpreted as the concentration of contaminant relative to background or upstream values.

For purposes of application, it is more convenient to express the cell concentration, C_n, in terms of W_0, the contaminant loading from the sludge disposal facility to the first cell of the cascade. This can be accomplished as follows. From Equation 5-1, the steady-state concentration in cell (n-1) of the cascade can be written as:

$$C_{n-1} = \frac{W_{n-2}}{Q_{fn-1} + A_{n-1} K_{totn-1}} \tag{5-2}$$

The loading of contaminant mass from cell (n-1) to cell n, or W_{n-1}, is found as the product of concentration and discharge,

$$W_{n-1} = C_{n-1} Q_{fn-1} \tag{5-3}$$

Combining Equations 5-2 and 5-3 with 5-1 gives:

$$C_n = \frac{W_{n-2}}{Q_{fn} + A_n K_{totn}} \; \frac{Q_{fn-1}}{Q_{fn-1} + A_{n-1} K_{totn-1}}$$

With (n-1) repetitions, this procedure leads to the following expression for contaminant concentration in the nth cell:

$$C_n = \frac{W_0}{Q_{fn} + A_n K_{totn}} \; \frac{Q_{fn-1}}{Q_{fn-1} + A_{n-1} K_{totn-1}} \cdots \frac{Q_{f1}}{Q_{f1} + A_1 K_{tot1}} \qquad (5\text{-}4)$$

With the introduction of a dimensionless cell "transport factor,"

$$\gamma_i = \frac{Q_{fi}}{Q_{fi} + A_i K_{toti}} \qquad (5\text{-}5)$$

and the definition of the cumulative cascade transport factor,

$$\Gamma_n = \prod_{i=0}^{n-1} \gamma_i \qquad (5\text{-}6)$$

Equation 5-4 can be written more compactly:

$$C_n = \frac{\Gamma_n W_0}{Q_{fn} + A_n K_{totn}} \qquad (5\text{-}7)$$

The parameters defined in Equation 5-7 have clear physical interpretations. The term γ_i represents the proportion of contaminant mass that enters cell i and is transported downstream to cell i+1. By definition, $\gamma_0=1$. Conversely, the quantity $1-\gamma_i$ is the percentage of contaminant mass that enters cell i and is degraded there. The weighting factor Γ_n gives the percentage of the initial contaminant load, W_0, that is transported downstream through the cascade to the nth cell in the sequence. The values γ_n for $n = 1,2,3...$ form a decreasing geometric type of series signifying the cumulative transport effect of the cascading cells model.

The cascading cells model given in Equation 5-7 is straightforward to apply yet sufficiently flexible to accommodate a wide variety of situations. The user need only define the cells constituting the cascade and then, for each cell, estimate three parameters: the river discharge (Q_f), the cell top surface area (A), and the net contaminant loss rate (K_{tot}). Further discussion of the determination of input parameters is given in Section 5.4.

The number of cells used depends on the physical conditions of the study reach and on the judgment of the modeler. As a general rule, individual cells should be established for reaches of the river over which conditions are reasonably uniform. Whenever there is an appreciable change in cross-sectional geometry, bed slope, river discharge or net contaminant loss rate, a new cell should be defined. This is particularly important with tributary confluences, where cell boundaries must be established to properly account for dilution effects. It is worthwhile to point out that the cascading cells model is the discrete analog of the classic continuous plug flow model. If any cell in the cascade were subdivided many times with transverse slices, the response for this "sliced" cell would approach the exponential response of the plug flow model. In fact, either approach (cascading cells or plug flow) could have been adopted for the surface water model.

One advantage of the model is the ease with which it can be used to calculate sludge criteria. That is, given a maximum allowable contaminant concentration at some downstream location (assumed to equal the RWC) the cascading cells model can determine the corresponding mass loading from the sludge disposal site to the river. As can be seen from Equation 5-7, the estimated concentration of contaminant in the last cell (C_n) is directly proportional to the estimated loading of contaminant to the stream. Rearranging Equation 5-7 yields a fixed ratio between estimated concentrations in the last cell of the stream, and estimated loadings from the aquifer:

$$C_n/W_0 = \frac{\Gamma_n}{(Q_{fn} + A_n K_{totn})} \qquad (5\text{-}8)$$

Since all terms on the right-hand side are known or can be estimated, Equation 5-8 provides a simple, direct method for estimating the maximum allowable loading of contaminant to the river (W_0) based on a maximum allowable value of C_n, which equals the RWC.

The final step in the derivation of criteria is to combine results from earlier calculations to yield maximum values for allowable contaminant concentrations in sludge accumulated or deposited in the lagoon. In this context it is useful to define a "source-receptor ratio" or SRR_{sw} that represents the ratio of expected contaminant concentrations at the selected downstream location to the dry-weight concentration of contaminant in sludge received by the facility. Since all of the calculations described thus far are linear with respect to contaminant concentrations:

$$SRR_{sw} = C_n/N = (C_n/W_0)(W_0/C_l)(C_l/N) \qquad (5\text{-}9)$$

where:

SRR_{sw} = source-receptor ratio, or the ratio of downstream concentrations of contaminant to dry-weight concentrations of contaminant in sludge (kg/*l*)

C_n = downstream contaminant concentration in the surface water (mg/*l*)

N = dry-weight concentration of the contaminant in sludge received by the impoundment (mg/kg)

W_0 = contaminant loading to the surface water (mg/sec)

C_l = contaminant concentration in liquid within the impoundment (mg/*l*)

The first term on the right side of Equation 5-9 can be calculated from Equation 5-8; the second term can be calculated with methods described in Section 4.3.1, and the third term can be calculated from Equation 4-1.

Equation 5-9 can be used to derive maximum allowable concentrations of a contaminant in sludge received by the surface disposal facility, as a linear function of the reference water concentrations for that contaminant:

$$N_{max} = RWC/SRR_{SW} \qquad (5\text{-}10)$$

where:

N_{max} = the maximum allowable dry-weight concentration of this contaminant for sludge received by this facility (mg/kg)

RWC = reference water concentration (mg/l)

Derivation of reference water concentrations for surface water will be discussed in Section 5.5. As an alternative approach, SRR_{SW} can be combined with other results to derive criteria on the basis of simultaneous exposure through more than one pathway. Consideration of multiple pathways of exposure will be discussed in Chapter 7.

5.4. INPUT PARAMETER REQUIREMENTS

In order to use the cascading cells model, the user must first define a series of cells; guidance for selecting the number and size of cascading cells has been provided in Section 5.3. Once the configuration of cells has been established, it remains for the user to specify three parameters for each cell: (1) the mean annual river flow, Q_f; (2) the top surface area, A; and (3) the net contaminant loss rate, K_{tot}.

5.4.1. Mean Annual River Flow. If the study reach is far from a gage, so that records of mean annual flow rates are not available, then a simple regression analysis using stream flow records from nearby rivers could be performed to estimate a regional relationship between discharge and watershed area. Based on the watershed area along the study reach, this regional relationship can provide an estimate of the mean annual flow.

For assessing potential risks to human health or wildlife from short-term, acute exposures to contaminants, the 7Q10, or minimum average 7-day flow expected to occur every 10 years, can be used in the equations in place of the mean annual flow rate. Values for these low-flow rates can be obtained from district offices of the U.S. Geological Survey, or from the GAGE data set accessible through the Graphical Exposure Modeling System (U.S. EPA, 1988c).

5.4.2. Cell Area. The top surface area of the cell, A, is computed as the product of the length, L, and the width, B, of each cell or,

$$A = L\ B \tag{5-11}$$

The cell length is the longitudinal distance between the upstream and downstream boundaries of the cell. The cell width is the hydraulic top width of the river when conveying the mean annual discharge. If possible, the top width measurement for each cell should be based on actual river cross sections obtained from field surveys. It is quite possible that surveyed cross sections along the study reach already exist as part of flood insurance studies performed for the U.S. Army Corps of Engineers, or the Federal Emergency Management Agency. This possibility should be explored before embarking on an expensive field survey program. If, however, there are no existing site-specific field data, then (as a minimum) cross sections should be obtained at the upstream and downstream boundaries of all anticipated cells. In principle, the top width of each cell can be found using a discharge rating curve and a typical cross section for each cell. For purposes of this analysis, however, it may be sufficient to approximate the cell top width as the transverse distance from left bank to right bank. If feasible, hydraulic field work should also include verification of Manning's roughness coefficient and a measurement of the suspended sediment concentration. On-site measurements can also help determine or verify several of the other key parameters (e.g. k_d, f_{oc}, and K_{tot}) to be discussed below.

5.4.3. Net Contaminant Loss Rate. It is convenient to express the net contaminant loss rate, K_{tot}, as the sum of two distinct loss mechanisms:

$$K_{tot} = K_d + k_s. \tag{5-12}$$

Here K_d represents the loss of the dissolved fraction due to all decay and exchange processes in the water column and k_s represents the loss due to interaction with the sediment.

As mentioned previously, it is assumed that the sediment bed is stationary and that storm events have a negligible effect on long-term average concentrations of contaminant in the stream for cases where interaction with sediment does occur. This assumption implies that $k_s = 0$, meaning that loss of contaminants to the stream bottom is not considered. This restriction has important ramifications for modeling heavy metals. If the sediment interaction pathway is eliminated, then the overall loss rate for heavy metals will be zero, since heavy metals neither decay nor volatilize. Stated another way, over the long term, there is no accumulation of heavy metals in the sediment bed. Instead, the total mass of the heavy metal transported in the water column is conserved. Hence, with the proposed modeling approach, dilution is the only mechanism affecting the total concentration of a heavy metal. It should be noted that under actual conditions, the sediment bed is likely to be removed, at least in part, by routine losses and flood events. By assuming that the sediment bed is stationary and at equilibrium, the methodology ignores contaminant losses through this route, leading to a conservative estimate of contaminant concentrations in the stream. Use of a more sophisticated model of contaminant transport in surface water (for example, the WASP4 model) allows the modeler to consider these processes in more detail, but does not seem warranted for the present application.

Given the suspended sediment concentration and the partition coefficient, the total concentration of a heavy metal in the water column can be resolved into dissolved and particulate fractions. Possible speciation associated with ionization, complexation or other reactions is ignored.

For toxic organic chemicals, it is helpful to express the dissolved contaminant loss rate as the weighted sum of two loss rates,

$$K_d = [k_{agg}H + k]f_d \qquad (5\text{-}13)$$

where:
K_d = dissolved contaminant loss rate (sec^{-1})
k_{agg} = "aggregate" decay rate (sec^{-1})
H = average depth of flow in the cell (m)
k = overall mass transfer coefficient for volatilization (m/sec)
f_d = dissolved fraction of the total mass of contaminant (unitless)

The fraction dissolved is given by:

$$f_d = \frac{1}{1 + k_d \, r_{sl}} \qquad (5\text{-}14)$$

where:
f_d = the fraction of contaminant mass in dissolved phase (unitless)
k_d = the partition coefficient of the linear isotherm describing the adsorption-desorption process at equilibrium (unitless)
r_{sl} = the ratio of the mass of solids to the volume of liquid in the flow (kg/*l*)

In surface water systems and other systems with a relatively low ratio of solids to liquid on a volume basis, r_{sl} can be approximated by the concentration of suspended sediment.

The partition coefficient for organic chemicals can be derived from organic carbon partition coefficients presented in U.S. EPA (1986a), can be estimated with the CHEMEST procedures in GEMS (U.S. EPA, 1988c), or can be estimated from octanol-water partition coefficients with the following empirical relation (Thomann and Mueller, 1987):

$$k_d = \frac{2.8(f_{oc}K_{ow})}{1.4 + r_{sl}(f_{oc}K_{ow})} \qquad (5\text{-}15)$$

where:
f_{oc} = weight fraction of organic carbon of the total solids concentration (unitless)
K_{ow} = octanol-water partition coefficient of the contaminant (unitless)

Values of K_{ow} and the corresponding k_d for 10 critical toxic organic chemicals are summarized in Table 5-2.

TABLE 5-2

Properties of Selected Organic Chemicals

Organic Chemical	$Log_{10}K_{ow}$ (unitless)	Partition Coefficient* (l/kg)	Henry's Law Constant (m^3-atm/mol)
Benzene	2.11-2.48	30	4.4×10^{-3}-5.5×10^{-3}
Benzo(a)pyrene	4.05-6.04	11,700	4.9×10^{-7}
BEHP	8.73	28,000	3.0×10^{-7}
Chlordane	2.78	120	2.8×10^{-6}
DDT	4.89-6.91	19,400	1.6×10^{-5}-4.8×10^{-5}
Dimethyl nitrosamine	0.06	0.23	3.3×10^{-5}
Lindane	3.11-3.72	620	7.5×10^{-5}-4.8×10^{-7}
PCBs (Aroclor 1248)	5.75-6.11	24,500	1.2×10^{-2}-2.4×10^{-3}
Toxaphene	3.30	390	2.1×10^{-1}
Trichloroethylene	2.29	40	8.8×10^{-3}

*From Equation (5-15) with $r_{sl}=10^{-4}$ kg/l and $f_{oc}=0.10$

The overall mass transfer coefficient (k) in Equation 5-13 is calculated based on a two-layer resistance model. Because the contaminant must pass through both liquid and air to be released into the atmosphere, the overall resistance equals the sum of the liquid and gas phase resistances. The overall mass transfer coefficient can thus be defined in terms of individual liquid and gas coefficients:

$$\frac{1}{k} = \frac{1}{k_l} + \frac{RT}{H_c k_g} \qquad (5\text{-}16)$$

where

k_l = the liquid phase mass transfer coefficient (m/sec),
k_g = the gas phase mass transfer coefficient (m/sec),
R = the ideal gas constant = 8.21×10^{-5} (m^3-atm/mol-K),
T = temperature (°K) and
H_c = Henry's law constant for contaminant (m^3-atm/mol).

Numerous methods for calculating k_l and k_g for water surfaces have been proposed in the literature (Hwang, 1985; Hwang and Thibodeaux, 1985; MacKay and Leinonen, 1975; MacKay and Yeun, 1983; Shen, 1982; Springer et al., 1984, U.S. EPA, 1987a; U.S. EPA, 1989c). This methodology follows an approach selected by U.S. EPA (1987a, 1989c) for estimating volatilization from surface impoundments. That approach, which should also be applicable to other surface water bodies, calculates mass transfer coefficients according to two types of site characteristics: (1) the ratio of the surface's effective diameter (or "fetch") to its depth and (2) the local average wind speed. Effective diameter is defined as $2(A/\pi)^{0.5}$, where A is the area of the water surface.

For sites at which the ratio of fetch:depth is less than 14 and for which local average wind speeds 10 meters above the surface (U_{10}) are greater than 3.25 m/sec, U.S. EPA (1987a) uses equations taken from MacKay and Yeun (1983) to calculate k_l. These equations are based on laboratory tests with a wind-wave tank, and estimate k_l as a function of the "friction velocity" of wind, defined as

$$U^* = 0.01\ U_{10}\ (6.1 + 0.63\ U_{10})^{0.5} \qquad (5\text{-}17)$$

where U_{10} is the average wind speed 10 meters above the liquid surface. If $U^* \geq 0.3$ m/sec, then

$$k_l = 1.0 \times 10^{-6} + 34.1 \times 10^{-4} U^* Sc_L^{-0.5} \tag{5-18}$$

otherwise, if $U^* < 0.3$,

$$k_l = 1.0 \times 10^{-6} + 144 \times 10^{-4} U^* 2.2 Sc_L^{-0.5} \tag{5-19}$$

where Sc_L equals the Schmidt number on the liquid side, defined as

$$Sc_L = \mu_w / (\rho_w D_{cw}) \tag{5-20}$$

and where:

μ_w = viscosity of water (g/cm-sec),
ρ_w = density of water (g/cm^3) and
D_{cw} = diffusivity of constituent in water (cm^2/sec).

For most cells likely to be specified to represent surface water bodies, the ratio of fetch:depth will be greater than 14. For water surfaces in areas where the fetch:depth ratio is greater than 14 or U_{10} is less than 3.25 m/sec, the methodology uses three different expressions for k_l taken from Springer et al. (1984) as reported in U.S. EPA (1987a). For all sites where $U_{10} < 3.25$ m/sec,

$$k_l = 2.78 \times 10^{-6} [D_{cw} / D_{ether}]^{2/3} \tag{5-21}$$

where D_{ether} is the diffusivity of ether in water (8.5×10^{-6} cm^2/sec). For those with $U_{10} \geq 3.25$ m/sec and fetch-to-depth ratios (FD) between 14 and 51.2,

$$k_l = [2.605 \times 10^{-9} FD + 1.277 \times 10^{-7}] U_{10}^2 [D_{cw}/D_{ether}]^{2/3} \tag{5-22}$$

where FD is the fetch:depth ratio. Finally, for facilities where $U_{10} \geq 3.25$ m/sec and FD ≥ 51.2,

$$k_l = 2.611 \times 10^{-7} U_{10}^{2} [D_{cw}/D_{ether}]^{2/3} \qquad (5\text{-}23)$$

Calculation of the mass transfer coefficient for the gas phase is based on Hwang (1985). For all values of FD and U^{10}, k_g (in m/sec) is calculated from

$$k_g = 1.8 \times 10^{-3} U_{10}^{0.78} Sc_G^{-0.67} d_e^{-0.11} \qquad (5\text{-}24)$$

where Sc_G equals the Schmidt number on the gas side, defined as

$$Sc_G = \mu_a / (\rho_a D_{ca}) \qquad (5\text{-}25)$$

and where:

μ_a = viscosity of air (g/cm-sec),
ρ_a = density of air (g/cm^3) and
D_{ca} = diffusivity of constituent in air (cm^2/sec).

Default values for μ_a and ρ_a (1.8×10^{-4} g/cm-sec and 1.2×10^{-3} g/cm-sec at STP, respectively) can be taken from Incropera and DeWitt (1985). Equations 5-16 through 5-25 are sufficient to estimate k, the overall mass transfer coefficient for the dissolved fraction of the contaminant.

The transformation processes of photolysis, hydrolysis and biodegradation are assumed to follow linear kinetic reactions. The combined effect of these transformations is represented with an "aggregate" loss rate, defined as:

$$k_{agg} = k_p + k_h + k_b \qquad (5\text{-}26)$$

where:

k_{agg} = aggregate loss rate (sec^{-1})
k_p = overall loss rate due to photolysis (sec^{-1})
k_h = overall loss rate due to hydrolysis (sec^{-1})
k_b = overall loss rate due to biodegradation (sec^{-1})

Estimates of organic chemical loss rates due to photolysis, hydrolysis and biodegradation are tabulated by Schnoor et al. (1987); biodegradation rates are also tabulated by Pitter, 1976. The overall loss rate due to biodegradation is sensitive to water temperature. When the temperature is different from 20°C, the value for k_b should be adjusted using:

$$[k_b]_T = [k_b]_{20}\Theta^{(T-20)} \tag{5-27}$$

where the temperature correction factor, Θ, is 1.06 (Thomann and Mueller, 1987) and the temperature is expressed in degrees Celsius.

5.4.4. Hydraulic Characteristics.

Several of the transfer rate expressions require estimates of hydraulic characteristics such as mean depth and/or mean velocity. If steady uniform flow is assumed for the length of the cell, the depth and velocity of the flow can be estimated using Manning's formula. If Q_f is the total flow through the cell expressed in metric units (i.e., cubic meters per second), then the discharge per unit width, q_d, is computed as:

$$q_d = \frac{Q_f}{B} \tag{5-28}$$

where:

q_d = discharge per unit width (m^2/sec)
Q_f = mean annual flow (m^3/sec)
B = width of cell (m)

and the depth of flow in the cell can be estimated with

$$H = \frac{(nq_d)^{0.6}}{S^{0.3}} \tag{5-29}$$

where:

H = mean depth of flow (m)
n = Manning's roughness coefficient (unitless)

S = longitudinal slope of the cell (unitless)

The longitudinal cell slope can be obtained from the bed slope of the river. Values for Manning's roughness coefficient vary considerably among streams. For natural streams, values range from 0.025-0.033 for smoothest beds, 0.045-0.060 for roughest beds, and 0.075-0.150 for very weedy streams (Thomann and Mueller, 1987). Guides for determining Manning's n have been compiled by Chow (1959) and Barnes (1967).

Combining Equations 5-28 and 5-29 gives the average flow velocity:

$$U_s = \frac{q_d}{H} \qquad (5\text{-}30)$$

The cascading cells model assumes well-mixed conditions in each cell. The model will better represent well-mixed conditions if the length of the cell, L, is greater than L_m, the length required for complete mixing. To check this assumption, a rough approximation of the distance L_m (measured from a side-bank discharge to the zone of complete mixing) can be obtained from:

$$L_m = 8.53 U_s \frac{B^2}{H} \qquad (5\text{-}31)$$

Where possible, cell lengths should be selected so that $L \geq L_m$.

5.4.5. Wind Speed. The wind speed (U_{10}) needed in Equations 5-17, and 5-22 through 5-24, can be obtained from Figure 5-2, which gives the prevailing direction and the mean annual speed of wind for sites around the nation (Environmental Sciences Service Administration, 1968).

5.5. HEALTH AND ENVIRONMENTAL EFFECTS

Contamination of surface water can cause adverse effects for both wildlife and humans:

1. Contaminants can cause adverse effects on fish and other biota inhabiting streams, lakes and estuaries. If the concentration of a particular pollutant

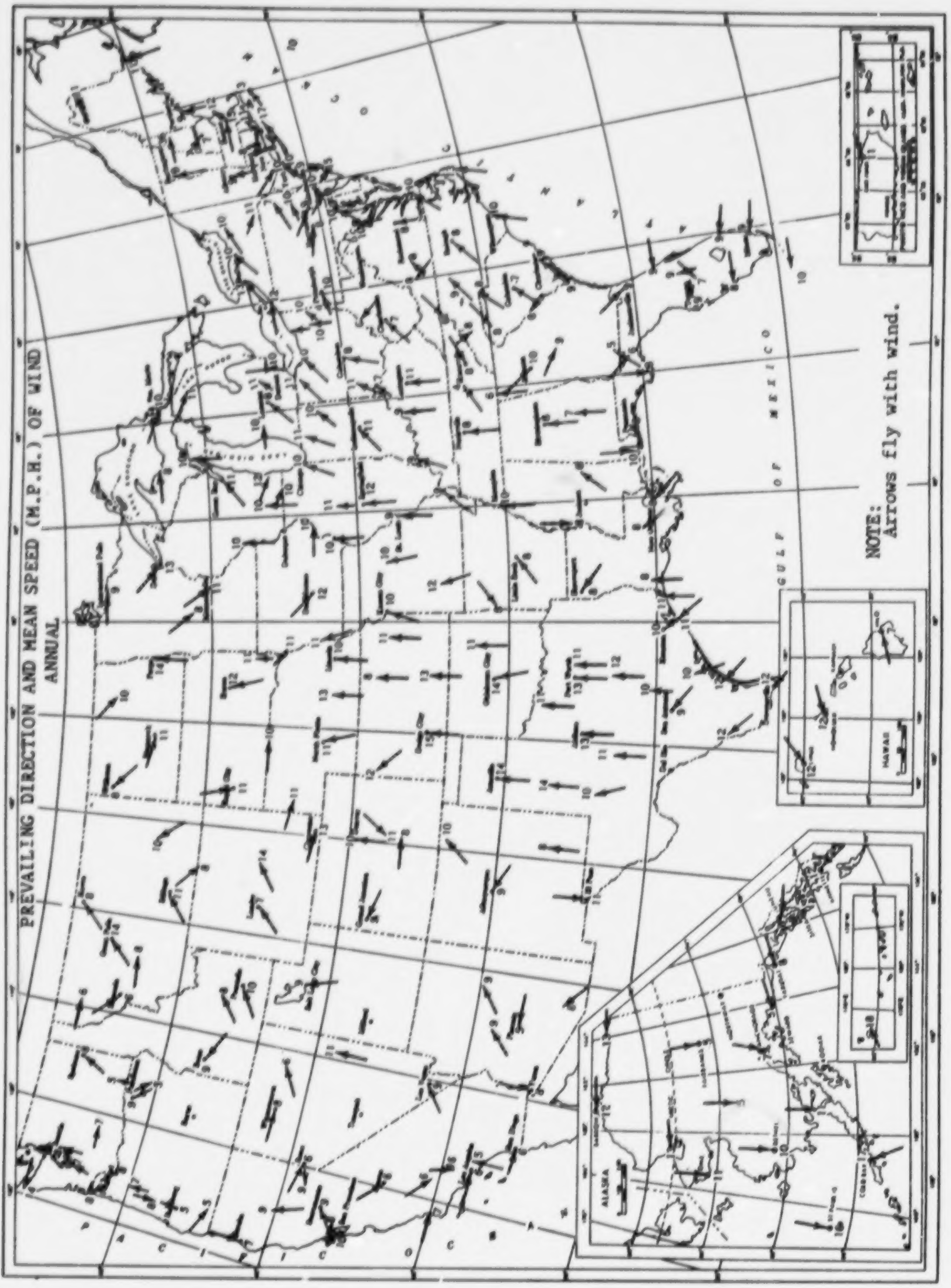

PREVAILING DIRECTION AND MEAN SPEED (M.P.H.) OF WIND
ANNUAL
NOTE:
Arrows fly with wind.
PACIFIC OCEAN
GULF OF MEXICO
ATLANTIC OCEAN
HAWAII
ALASKA

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FIGURE 5-2

Mean Annual Wind Speed in the United States exceeds threshold levels, fish and other biota may die or suffer adverse health or reproductive effects.

2. Adverse effects on human health can be direct (by water consumption) or indirect (by fish or shellfish consumption). If the human diet includes fish or shellfish that have bioconcentrated a toxic pollutant from surface waters, the indirect mode of toxicant consumption may pose more risk than the direct mode.

The first step in deriving criteria for surface disposal of sludge is the derivation of a reference water concentration (RWC) for each contaminant, below which adverse effects are not expected. Such reference concentrations should be protective of both the environment (e.g., aquatic life) and human health.

5.5.1. Aquatic Life Protection. Protection of aquatic life from long-term effects should be based on the Ambient Water Quality Criteria (U.S. EPA, 1980c). The AWQC consist of two concentrations: criteria for chronic exposures that should not be exceeded as a yearly average; and criteria for acute exposures that should not be exceeded, on average, in a 24-hour period. Estimated yearly average concentrations should be compared with criteria for chronic exposures, and estimates based on 7Q10 flows should be compared with acute criteria. For chemicals for which criteria are not available, the literature should be evaluated to determine whether useful data have become available since the AWQC were developed.

5.5.2. Threshold-Acting Toxicants. If the only source of potential exposure is drinking water, then the RWC for a threshold-acting toxicant is derived with methods described in Section 4.5.1. If the only source of potential exposure is fish living in contaminated surface waters, then the RWC is calculated as:

$$\mathrm{RWC} = [(\mathrm{RfD\ BW\ RE}^{-1}) - \mathrm{TBI}] / (\mathrm{BCF}\ I_f) \qquad (5\text{-}32)$$

where:

RWC = reference water concentration (mg/l)
RfD = reference dose (mg-day/kg)
BW = human body weight (kg)
RE = relative effectiveness of ingestion exposure (unitless)
TBI = total background intake rate of contaminant from all other sources of exposure (mg/day)
BCF = unadjusted bioconcentration factor in fish (l/kg)
I_f = total rate of fish ingestion (kg/day)

If humans are exposed to sludge contaminants through both drinking water and consumption of fish, the reference concentration is calculated as:

$$RWC = [(RfD\ BW\ RE^{-1}) - TBI] / [I_w + (BCF\ I_f)]$$

where:

I_w = total water ingestion rate (*l*/day)

The definition and derivation of bioconcentration factors and fish ingestion rates are provided below. Other parameters in Equation 5-32 are described in Section 4.5.

5.5.2.1. BIOCONCENTRATION FACTOR (BCF) -- Bioconcentration is the ability of living organisms to accumulate substances to higher than ambient level concentrations. The degree to which a chemical accumulates in an aquatic organism above ambient concentrations is indicated by the bioconcentration factor (BCF), which is defined as the quotient of the concentration of a substance in all or part of an aquatic organism (mg/kg fresh weight) divided by the concentration in the water to which the organism has been exposed (mg/*l*). The BCF is usually determined at equilibrium conditions, or for 28-day exposures, is based upon the fresh weight of the organism, and is specified in units of mg/kg per mg/*l*, or *l*/kg (or unitless, since 1*l* of water has a mass of 1 kg). Other terms are used to describe increases of environmental pollutant in organisms, such as biomagnification, bioaccumulation or ecological magnification. Bioconcentration is distinguished from these other processes in that BCF considers only the uptake of contaminant from physical surroundings (i.e., the contaminated water in which the organism lives) and does not consider uptake from food sources. Although BCF has been documented as the primary pathway for accumulation in numerous studies (Marcelle and Thome, 1984; Bahner et al., 1977; Clayton et al., 1977), there is also evidence that biomagnification by aquatic food chains can be important under certain environmental circumstances (Lee et al., 1976).

Bioconcentration factors are specific for the pollutant and for the species absorbing the pollutant. Pollutants that are lipophilic and resistant to biodegradation are most likely to bioconcentrate in living organisms. Initial diffusion into the organism occurs by rapid surface adsorption or partitioning to the lipoprotein layer of cell membranes. Once in the bloodstream, subsequent accumulation of the pollutant into particular compartments of the organism is dependent on the metabolic capabilities of the species and the lipid content of the individual organism. With continued exposure, an equilibrium condition is eventually reached where the rate of contaminant excretion is equal to the rate of uptake.

Bioconcentration factors can be estimated through laboratory experiments, field studies, correlations with physicochemical factors such as octanol-water partition coefficients, and models based on pollutant biokinetics coupled with fish energetics. In developing the ambient water quality criteria, the U.S. EPA used laboratory data in the calculation of BCFs. Field data are often less useful than laboratory data because of the difficulty of establishing the field pollutant concentrations over time and of determining the range of territory inhabited by the organism. BCFs calculated from field data tend to be greater than those calculated from laboratory data because they may also include ingestion of the pollutant through the consumption of prey, sediment and water, as well as direct absorption from water.

Where laboratory and field data are not available, bioconcentration factors can be estimated by several methods. Correlations between BCFs and octanol-water partition coefficients, water solubility and soil adsorption coefficients have been documented. Veith et al. (1979) developed the following equation using the correlation between BCF and the (unitless) octanol water partition coefficient (K_{OW}):

$$\log_{10} \text{BCF} = 0.85 \log_{10} K_{OW} - 0.70$$

Using this equation, Veith et al. were able to estimate BCFs to within 60% before laboratory testing. The equation was developed using data from the whole-body analyses of fish with approximately 7.6% lipids (U.S. EPA, 1980c). The U.S. EPA adopted the equation for use in determining BCFs in the exposure sections of the health effects chapters of the Ambient Water Quality Criteria Documents in cases where the BCF was not available from other data. In a later study, Veith et al. (1980) used the results of their own laboratory experiments and data from other laboratories for a variety of fish species and 84 different organic chemicals to obtain the following modification of their original equation:

$$\log_{10} \mathrm{BCF} = 0.76 \log_{10} K_{ow} - 0.23$$

Equations similar to the ones developed by Veith et al. (1979, 1980) have been developed for more specific chemical classes and particular aquatic species (Veith et al., 1979; Neeley et al., 1974). Other investigators (Norstrom et al., 1976) have developed more elaborate models using pollutant biokinetics and fish energetics in addition to using octanol-water partition coefficients to predict bioconcentration factors.

Since bioconcentration for lipophilic compounds is based on the lipid content of the organism, the estimated or measured BCF for these lipophilic compounds must be adjusted by the average lipid content of seafood consumed in the U.S. diet. In 1980, the U.S. EPA determined that the average lipid content of the freshwater and estuarine species, weighted by average daily consumption, was about 3% (U.S. EPA, 1980c). Since fresh and estuarine waters would be affected by surface water contamination from surface disposal sites, a lipid content of 3% should be assumed when deriving the reference water concentration. The bioconcentration factor should be adjusted as follows:

$$\mathrm{BCF}_a = \mathrm{BCF}\,(\mathrm{LC}_d/\mathrm{LC}_e)$$

where:

BCF_a = adjusted bioconcentration factor (*l*/kg)
BCF = unadjusted bioconcentration factor (*l*/kg)
LC_d = lipid content of dietary seafood (kg/kg)
LC_e = lipid content of experimental organism (kg/kg)

5.5.2.2. FISH CONSUMPTION RATE (I_f) -- The most recent fish consumption document from the U.S. Department of Commerce (1985) reports that total per capita fish and shellfish consumption ranged from 12.8 pounds per year in 1980 to 13.6 pounds per year in 1984, as shown in Table 5-3. The latter value corresponds to a daily intake of 16.9 grams of fish (all kinds). These figures do not include recreational catch, of which average consumption is estimated to be an additional 3-4 pounds per year or 3.7-5 grams per day (U.S. EPA, 1980a). If 3.5 pounds per year, or 4.4 grams per day, is consumed from recreational fishing, the total average per capita intake of all types of seafood is about 21.3 grams per day.

Surface water contamination from surface disposal sites could affect freshwater and estuarine species, but is unlikely to affect the marine species that constitute a substantial portion of seafood in the U.S. diet. To estimate the average daily consumption of freshwater and estuarine species, the U.S. EPA analyzed data from a survey of fish consumption in 1973-74 (as reanalyzed by U.S. EPA, 1980a) and eliminated all species not taken from fresh or estuarine waters (Stephan, 1980). Per capita consumption of these species was estimated to be 6.5 grams per day, or about half the estimate of total fish consumption derived from that study, 13.4 grams per day. Based on this information, it is reasonable to conclude that fresh and estuarine species constitute about 50% of the total fish consumption in the U.S. diet. Combining this assumption with an estimated average total consumption of fish of 21.3 grams yields an estimate of daily consumption of freshwater and estuarine species of about 10.6 grams.

It should be noted that average daily rates of fish consumption vary substantially by region, age, race and religion. An analysis of fish consumption data by SRI International found that per-capita fish intake by black and Jewish populations to be double the average value for the total U.S. population (U.S. EPA, 1980a). The New England and East South Central regions of the United States had the highest regional rates of fish consumption. Consumption levels at the upper 95th percentile of the national distribution were 300-400% of the national average. The Asian population in the United States showed the highest 95th percentile consumption rate; this value was 502% of the reported national average.

TABLE 5-3

U.S. Annual Per Capita Consumption of Commercial Fish and Shellfish, 1960-1984[a,b]

Year	Civilian Resident Population (million persons)	Per Capita Consumption (pounds of edible meat)			Total
		Fresh & Frozen[c]	Canned[d]	Cured	
1960	178.1	5.7	4.0	0.6	10.3
1961	181.1	5.9	4.3	0.5	10.7
1962	183.7	5.8	4.3	0.5	10.6
1963	186.5	5.8	4.4	0.5	10.7
1964	189.1	5.9	4.1	0.5	10.5
1965	191.6	6.0	4.3	0.5	10.8
1966	193.4	6.1	4.3	0.5	10.9
1967	195.3	5.8	4.3	0.5	10.6
1968	197.1	6.2	4.3	0.5	11.0
1969	199.1	6.6	4.2	0.4	11.2
1970	201.9	6.9	4.5	0.4	11.8
1971	204.9	6.7	4.3	0.5	11.5
1972	207.5	7.1	4.9	0.5	12.5
1973	209.6	7.4	5.0	0.4	12.8
1974	211.6	6.9	4.7	0.5	12.1
1975	213.8	7.5	4.3	0.4	12.2
1976	215.9	8.2	4.2	0.5	12.9
1977[e]	218.1	7.7	4.6	0.4	12.7

TABLE 5-3 (cont.)

Year	Civilian Resident Population (million persons)	Per Capita Consumption (pounds of edible meat)			Total
		Fresh & Frozen[c]	Canned[d]	Cured	
1978[e]	220.5	8.1	5.0	0.3	13.4
1979[e]	223.0	7.8	4.8	0.4	13.0
1980[e]	225.6	8.0	4.5	0.3	12.8
1981[e]	227.7	7.8	4.8	0.3	12.9
1982[e]	229.9	7.7	4.3	0.3	12.3
1983[e]	232.0	8.0	4.8	0.3	13.1
1984[e]	234.0	8.3	5.0	0.3	13.6

[a] Source: U.S. EPA, 1989d. Adapted from U.S. Department of Commerce, 1985

[b] These consumption estimates refer only to fish and shellfish entering commercial channels, and they do not include data on consumption of recreationally caught fish and shellfish, which since 1970 is estimated to be between 3 and 4 pounds (edible meat)/person annually. The figures are calculated on the basis of raw edible meat (e.g., excluding bones, viscera, shells). The U.S. Department of Agriculture (USDA) consumption figures for red meats and poultry are based on the retail weight of the products, as purchased in retail stores. The USDA estimates are the net edible weight to be ~70-95% of the retail weight, depending on the cut and type of meat. From 1970 through 1980, data were revised to reflect the results of the 1980 census.

[c] Beginning in 1973, data include consumption of artificially cultivated catfish.

[d] Based on production reports, packer stocks and foreign trade statistics for individual years.

[e] Domestic landings data used in calculating these data are preliminary.

Applying the same percentage increase to an estimated average consumption rate of 10.6 grams per day yields an estimated consumption rate of about 53 grams of freshwater fish per day, or 43 pounds per year. Rates of fish consumption by various demographic variables are listed in Table 5-4.

5.5.3. Carcinogens. Section 4.5.2 describes an approach for deriving reference water concentrations for carcinogens in drinking water. If the only source of pollutant exposure is ingestion of contaminated fish, then the RWC is derived as:

$$RWC = (RL\ BW)/(q_1^*\ RE\ BCF\ I_f)$$

where:

q_1^* = upper bound human cancer potency (kg/mg-day)
RL = upper bound risk level (unitless)

If sources of human exposure include both ingestion of water from the affected surface water body and consumption of fish that live in the affected surface water body, then the RWC is calculated as:

$$RWC = \frac{(RL\ BW)}{(q_1^*\ RE)[I_w + (BCF\ I_f)]}$$

This method for deriving the RWC for carcinogens is thought to be conservative, because the human cancer potency value (q_1^*) is based on a 95th percentile confidence interval. All parameters have been defined in previous sections.

5.6. SAMPLE CALCULATIONS

5.6.1. Analysis of Exposure for the Most Exposed Individual. Use of the proposed surface water quality model is illustrated with sample calculations for one organic chemical (benzene) and one heavy metal (lead) at three locations in the United States: the Contoocook River near Antrim, New Hampshire, Bird Creek near Tulsa, Oklahoma, and the Columbia Slough near Portland, Oregon.

Table 5-5 provides input parameters used to describe the chemical characteristics of benzene and lead. As can be seen from the table, zero rates of photolysis and hydrolysis

TABLE 5-4

Fish Consumption by Demographic Variables*

Demographic Category	Mean Consumption (g/day)	Upper 95th Percentile (g/day)
Race:		
Caucasian	14.2	41.2
Black	16.0	45.2
Oriental	21.0	67.3
Other	13.2	29.4
Sex:		
Female	13.2	38.4
Male	15.6	44.8
Age (years):		
0-9	6.2	16.5
10-19	10.1	26.8
20-29	14.5	38.3
30-39	15.8	42.9
40-49	17.4	48.1
50-59	20.9	53.4
60-69	21.7	55.4
70+	13.3	39.8
Census Region:		
New England	16.3	46.5
Middle Atlantic	16.2	47.8
East North Central	12.9	36.9
West North Central	12.0	35.2
South Atlantic	15.2	44.1
East South Central	13.0	38.4
West South Central	14.4	43.6
Mountain	12.1	32.1
Pacific	14.2	39.2

*Source: U.S. EPA, 1989d. Adapted from U.S.EPA, 1980a

TABLE 5-5

Chemical Properties of Benzene and Lead

Chemical Property	Benzene	Lead	(Units)
K_{ow}	135	--	(unitless)
k_d	*	234	(l/kg)
H_c	5.5×10^{-3}	--	(atm-mol/m^3)
k_b	1.7×10^{-6}	0.0	(sec^{-1})
k_p	0.0	0.0	(sec^{-1})
k_h	0.0	0.0	(sec^{-1})
D_{ca}	8.8×10^{-2}	--	(cm^2/sec)
D_{cw}	9.8×10^{-6}	--	(cm^2/sec)
μ_w	1.8×10^{-4}	--	(kg/l)
RWC (drinking)	1.2×10^{-3}	5.0×10^{-3}	(mg/l)
RWC (fish)	8.8×10^{-3}	--	(mg/l)

*Varies according to suspended solids concentration and fraction of organic carbon in each stream have been assumed for both contaminants. Of the remaining parameters, only the partition coefficient is relevant to model calculations for lead. As in Chapter 4, Lead is assumed to have a partition coefficient of 234 l/kg, appropriate for sand (U.S. EPA, 1986a). Partition coefficients for benzene are calculated separately for each site with Equation 5-15, based on the octanol-water partition coefficient and assumed values for f_{oc} and r_{sl}. Most of the hydraulic input parameters for the sample calculations have been derived from site-specific data and the U.S. Geological Survey (1982, 1983, 1988a); water quality parameters have been obtained from available literature. The following sections present a Tier 2 analysis for these sites.

5.6.1.1. SAMPLE CALCULATION: ANTRIM, NEW HAMPSHIRE -- This study area encompasses a 25-kilometer reach of the Contoocook River between the towns of Antrim and Hillsboro in southern New Hampshire. The Antrim wastewater treatment plant accumulates and stores sludge in three impoundments located next to the Contoocook River. Hillsboro was selected as the location of the MEI since it is the first community downstream from the sludge impoundment that would use the Contoocook River as a source of water.

5.6.1.1.1. Defining Cells for the Cascading Cells Model -- To represent contaminant transport in the Contoocook River, the study reach was divided into three cells as follows:

- Cell 1 extends from the Antrim sludge impoundment downstream for about 17 kilometers to the confluence of Beards Brook;
- Cell 2 extends from Beards Brook downstream for about 2.4 kilometers to the Hillsboro Paper Company dam;
- Cell 3 extends from the Hillsboro dam for about 5.1 kilometers to the downstream limits of Hillsboro.

The process of selecting these cells was guided by the dual objectives of minimizing the total number of cells in the cascade, and yet maintaining uniform geometry and discharge through each cell. Selection of the final cascade configuration was based on a review of the longitudinal profile of the Contoocook River (Figure 5-3), river cross-

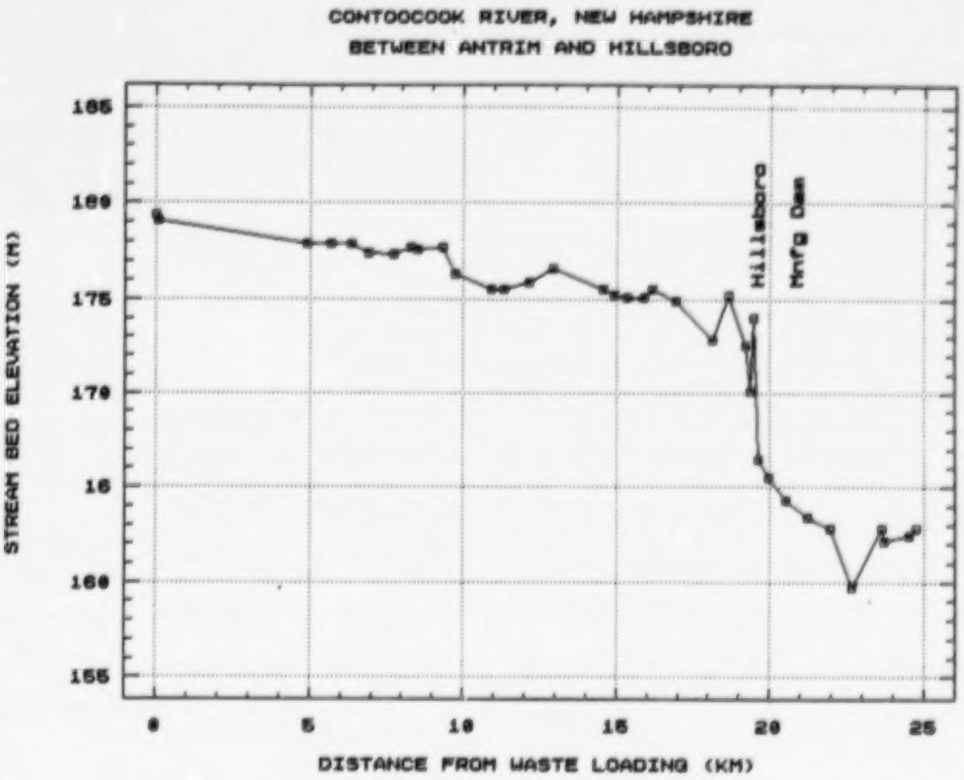


FIGURE 5-3

Longitudinal Profile of Contoocook River Study Reach sections, and topographic quad maps showing tributary inflows along the entire study reach. A proportional sketch of the three-cell cascade is shown in Figure 5-4.

To apply the cascading cells model given in Equations 5-7 and 5-8 to the configuration shown in Figure 5-4, it is necessary to estimate the three parameters Q_f, A and K_{tot} for each cell. Estimates of the mean annual discharge, Q_f, for the main stem of the Contoocook River and for its tributaries were obtained from the U.S. Geological Survey district office in Concord, New Hampshire. Flow values for all three cells are summarized in Table 5-6. As expected, the mean annual flow increases in the downstream direction.

The top surface area of each cell, A, was computed with Equation 5-11 after first measuring the width, B, of a rectangular channels fitted onto representative cross-sections of the Contoocook River, as shown in Figure 5-5 for Cell #2 of the cascade. At Cell 2, for example, with length $L_2 = 2410$ meters and an estimated top width of $B_2 = 36.6$ meters (from Figure 5-5), Equation 5-11 gives the top surface area for Cell 2 as:

$$A_2 = (L_2)(B_2) = (2,410)(36.6) = 88,200 \text{ m}^2$$

With a mean annual discharge $Q_{f2} = 17.1 \text{ m}^3/\text{sec}$ and a cell width of $B_2 = 36.6$ m, the discharge per unit width for Cell 2 is given by Equation 5-28:

$$q_{d2} = \frac{Q_{f2}}{B_2} = \frac{17.1}{36.60} = 0.47 \text{ m}^2/\text{sec}$$

Substituting this result into Equation 5-29, with an assumed Manning's $n_2 = 0.06$ and a measured bed slope $S_2 = 0.00027$, gives the depth of uniform flow in Cell 2 as:

$$H_2 = \frac{(n_2 q_{d2})^{0.6}}{S_2^{0.3}} = \frac{[(0.06)(0.47)]^{0.6}}{(0.00027)^{0.3}} = 1.37 \text{ m}$$

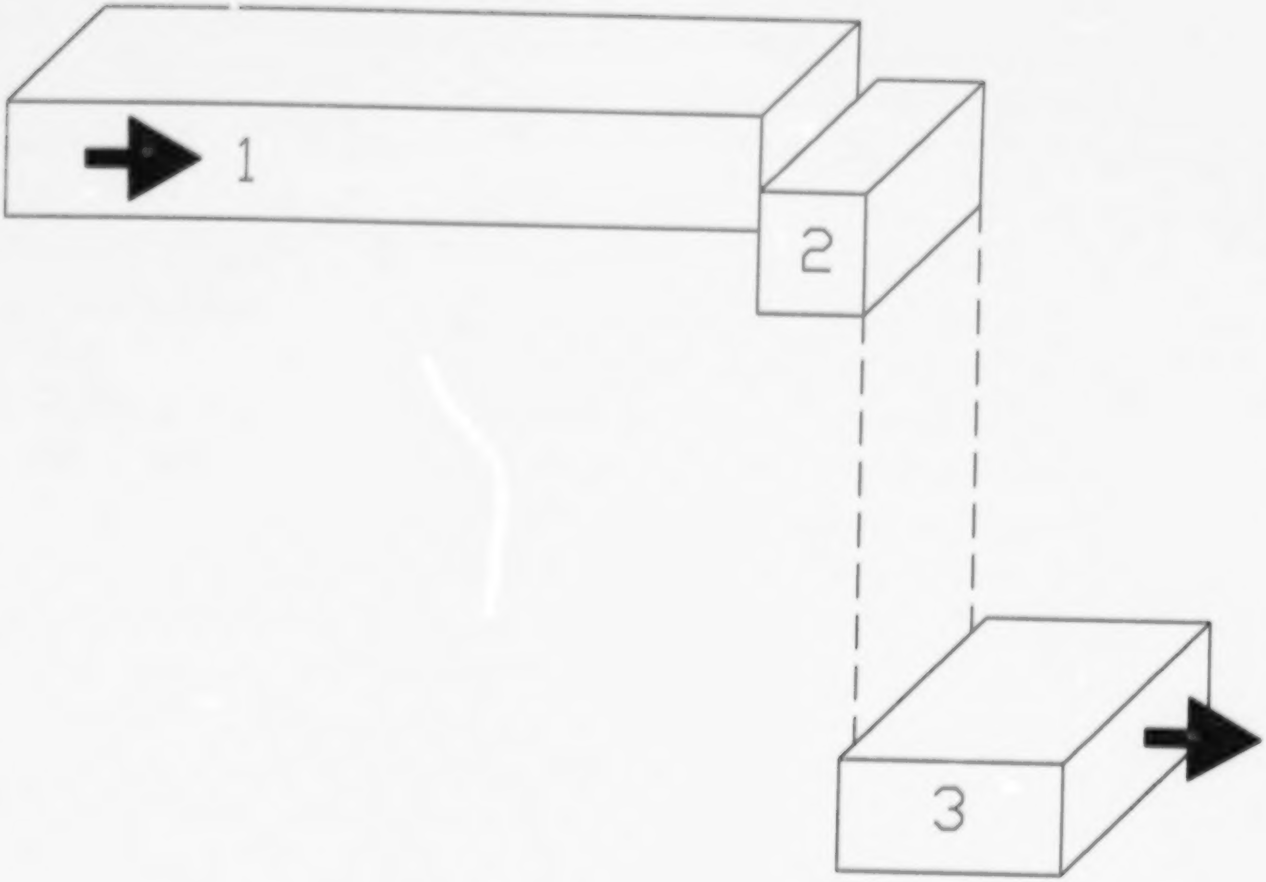


FIGURE 5-4

Three Cell Cascade for Contoocook River
Between Antrim and Hillsboro, NH

TABLE 5-6

Site-Specific Input Parameters
Contoocook River, New Hampshire

Parameter	Cell 1	Cell 2	Cell 3	(Units)
L	16,890	2410	5077	(m)
B	30.5	36.6	45.7	(m)
H	1.27	1.37	1.24	(m)
A	515,000	88,200	232,000	(m^2)
V	654,000	121,000	287,000	(m^3)
Q_f	12.6	17.1	18.0	(m^3/sec)
q_d	0.41	0.47	0.39	(m^2/sec)
U_s	0.32	0.34	0.31	(m/sec)
n	0.06	0.06	0.06	(unitless)
S	0.00027	0.00027	0.00027	(unitless)
L_m	2000	2800	4500	(m)
U_{10}	3.2	3.2	3.2	(m/sec)
T	25	25	25	(°C)
r_{sl}	$5x10^{-5}$	$5x10^{-5}$	$5x10^{-5}$	(kg/*l*)
f_{oc}	0.18	0.18	0.18	(unitless)

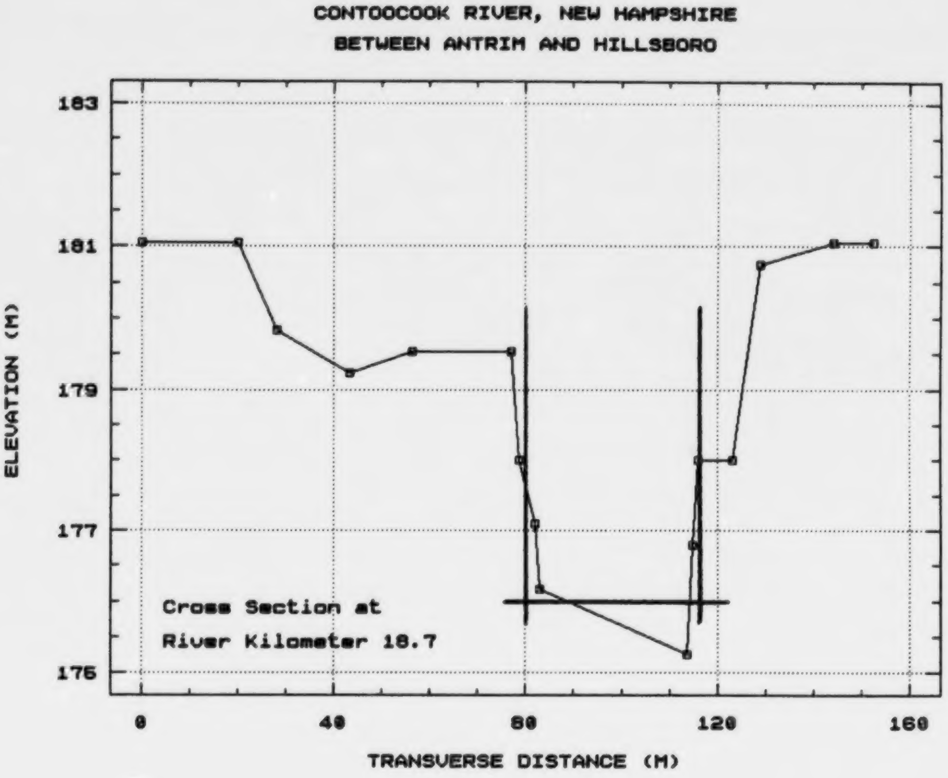


FIGURE 5-5

Typical Cross Section of Contoocook River

Substituting the estimated values for q_{d2} and H_2 into Equation 5-30 gives the flow velocity through Cell 2:

$$U_{s2} = \frac{q_{d2}}{H_2} = \frac{0.47}{1.37} = 0.34 \text{ m/sec}$$

These calculations are performed for all three cells of the cascade; the resulting hydraulic input parameters for the Contoocook River are summarized in Table 5-6.

From Equation 5-31 with $U_{s1} = 0.32$ m/sec, $B_1 = 30.5$ m, and $H_1 = 1.27$ m, the approximate distance downstream from the "point" of contaminant loading to the zone of complete mixing is:

$$L_{m1} = (8.53)(0.32)\frac{(30.5)^2}{1.27} = 2000 \text{ m}$$

Since $L_{m1} = 2000 < L_1 = 16{,}890$ m, the contaminant should be reasonably well-mixed in the cross section before it moves from Cell 1 to Cell 2.

The mean annual wind velocity at the site is estimated from Figure 5-2 to be $U_{10} = 3.2$ m/sec. The surface water temperature is taken to be $T = 25$ °C. The Department of Environmental Services in Concord, New Hampshire, has measured the suspended solids concentration in the Contoocook River at $5.0\text{x}10^{-5}$ kg/l along the study reach; this value is used for r_{sl}. Estimates for U_{10}, T and r_{sl} are assumed to be the same for all three cells in the cascade and are listed in Table 5-6.

5.6.1.1.2. Estimating Net Contaminant Loss Rates -- Determination of the net contaminant loss rate, K_{tot}, for benzene requires application of Equations 5-12 through 5-15 using a four-step procedure:

1. Estimate f_d, the dissolved fraction of benzene.
2. Estimate k, the overall mass transfer rate for benzene.

3. Estimate k_{agg}, the "aggregate" decay rate for benzene.
4. Estimate K_{tot}, the net contaminant loss rate for benzene.

From Table 5-2, the $\log_{10}(K_{ow})$ is taken as 2.13 and, hence, the octanol-water partition coefficient for benzene is $K_{ow} = 135$. The suspended solids concentration in the study reach is estimated to be $r_{sl} = 5.0 \times 10^{-5}$ kg/*l*. It is also reported by the New Hampshire Department of Environmental Services that the concentration of suspended organic carbon is 9×10^{-6} kg/*l*, so the weight fraction of organic carbon is $f_{oc} = (9 \times 10^{-6}/5 \times 10^{-5}) = 0.18$. Substituting these values for K_{ow}, f_{oc} and r_{sl} into Equation 5-15 gives,

$$k_d = \frac{2.8(f_{oc}K_{ow})}{1.4 + r_{sl}(f_{oc}K_{ow})}$$

$$= \frac{(2.8)(0.18)(135)}{1.4 + (5 \times 10^{-5})(0.18)(135)}$$

$$= 48.6 \ l/\text{kg}$$

Substituting this result into Equation 5-14 gives the dissolved fraction,

$$f_d = \frac{1}{1 + k_d \, r_{sl}}$$

$$f_d = \frac{1}{1 + (48.6)(5 \times 10^{-5})}$$

$$= 0.9976$$

which indicates that virtually all (i.e., 99.76%) of the benzene will be in solution in the water column. This result is not surprising, since the values for both k_d and r_{sl} are relatively low.

Derivation of the overall mass transfer coefficient k for Equation 5-13 requires Henry's constant and the two individual mass transfer coefficients k_g and k_l. From Table

5-2, Henry's constant for benzene is assumed to be 0.0055 atm-mole/m^3. The liquid film coefficient is estimated with Equation 5-21 for the wind velocity of 3.20 m/sec, D_{cw} = 9.8x10^{-6} cm^2/sec, and D_{eth} = 8.5x10^{-6} cm^2/sec:

$$\begin{aligned} k_l &= 2.78\times10^{-6}(D_{cw}/D_{eth})^{2/3} \\ &= 2.78\times10^{-6}\ (9.8\times10^{-6}/8.5\times10^{-6})^{2/3} \\ &= 3.1\times10^{-6}\ \text{m/sec} \end{aligned}$$

The Schmidt number for benzene is calculated from Equation 5-25 as:

$$\begin{aligned} Sc_g &= \mu_a/(\rho_a\ D_{ca}) \\ &= (1.8\times10^{-4})/[(1.2\times10^{-3})(8.8\times10^{-2})] \\ &= 1.70 \end{aligned}$$

Next, the mass transfer coefficient for gas is estimated from Equation 5-24 with an effective distance estimated as $(2)(88{,}200/\pi)^{0.5} = 355$ m:

$$\begin{aligned} k_g &= 1.8\times10^{-3}\ U_{10}{}^{0.78}\ Sc_G{}^{-0.67}\ d_e{}^{-0.11} \\ &= 1.8\times10^{-3}(3.2)^{0.78}(1.70)^{-0.67}(355)^{-0.11} \\ &= 1.6\times10^{-3}\ \text{m/sec} \end{aligned}$$

Finally with H_c=5.5x10^{-3} atm-m^3/mol, R=8.21x10^{-5} atm-m^3/mol-K, and T=298° K, the overall mass transfer rate for Cell 2 is found using Equation 5-16:

$$\begin{aligned} \frac{1}{k_2} &= \frac{1}{k_l} + \frac{RT}{H_c k_g} \\ &= \frac{1}{3.1\times10^{-6}} + \frac{(8.21\times10^{-5})(298)}{(5.5\times10^{-3})(1.6\times10^{-3})} \end{aligned}$$

which gives $k_2 = 3.0\mathrm{x}10^{-6}$ m/sec, as listed in Table 5-7.

The aggregate decay rate, k_{agg}, represents the summed effects of contaminant loss due to photolysis, hydrolysis and biodegradation. Of these three decay mechanisms, only biodegradation appears to significantly impact benzene (Schnoor et al. 1987). The benzene decay rate at 20°C due to biodegradation is $k_b = 1.27\mathrm{x}10^{-6}\ \mathrm{sec}^{-1}$. With Equation 5-27 and $\Theta = 1.06$, this loss rate is corrected for the assumed temperature of the river (25°C):

$$\begin{aligned}[k_b]_T &= [k_b]_{20}\Theta^{(T-20)}\\ [k_b]_{25} &= 1.27\mathrm{x}10^{-6}(1.06)^{(25-20)}\\ &= 1.70\mathrm{x}10^{-6}\ \mathrm{sec}^{-1}\end{aligned}$$

With $k_p = 0.0$, $k_h = 0.0$ and k_b given above, Equation 5-26 gives the aggregate decay rate for benzene as $k_{agg} = 1.7\mathrm{x}10^{-6}\ \mathrm{sec}^{-1}$.

The overall net loss rate for benzene is now computed with Equation 5-12. In Cell 2, with $H_2 = 1.37$ m, $f_d = 0.9976$, $k = 3.0\mathrm{x}10^{-6}$ m/sec and $k_{agg} = 1.7\mathrm{x}10^{-6}\ \mathrm{sec}^{-1}$, Equation 5-12 gives,

$$\begin{aligned}K_{tot} &= K_d + k_s\\ &= [(1.7\mathrm{x}10^{-6})(1.4)+(3.0\mathrm{x}10^{-6})](0.9976)\\ &= 5.4\mathrm{x}10^{-6}\ \mathrm{m/sec}\end{aligned}$$

The four steps outlined above are applied to all three cells in the cascade. Results for benzene are summarized in Table 5-7.

These steps are not needed for heavy metals. As mentioned previously, it is assumed that heavy metals neither decay nor volatilize. Further, it is assumed that the sediment bed does not act as a source or a sink of heavy metals. Hence the net loss rate for lead in the Contoocook River is zero as summarized in Table 5-8. The only parameter to be estimated for lead, then, is the partition coefficient, which in this case is taken to be

TABLE 5-7

Model Results for Benzene
Contoocook River, New Hampshire

Parameter	Cell 1	Cell 2	Cell 3	(Units)
k_d	48.6	48.6	48.6	(*l*/kg)
f_d	0.9976	0.9976	0.9976	(unitless)
k_l	$3.1x10^{-6}$	$3.1x10^{-6}$	$3.1x10^{-6}$	(m/sec)
k_g	$1.5x10^{-3}$	$1.6x10^{-3}$	$1.6x10^{-3}$	(m/sec)
k	$3.0x10^{-6}$	$3.0x10^{-6}$	$3.0x10^{-6}$	(m/sec)
k_{agg}	$1.7x10^{-6}$	$1.7x10^{-6}$	$1.7x10^{-6}$	(sec^{-1})
K_{tot}	5.2x10-6	$5.4x10^{-6}$	$5.1x10^{-6}$	(m/sec)
γ	0.82	0.97	0.94	(unitless)
Γ	1.0	0.82	0.80	(unitless)
C/W_0	$6.6x10^{-2}$	$4.7x10^{-2}$	$4.2x10^{-2}$	(sec/*l*)
C_d/W_0	$6.5x10^{-2}$	$4.7x10^{-2}$	$4.2x10^{-2}$	(sec/*l*)
W_0/N	$1.0x10^{-3}$	$1.0x10^{-3}$	$1.0x10^{-3}$	(kg/sec)
SRR_{SW}	$6.6x10^{-5}$	$4.7x10^{-5}$	$4.2x10^{-5}$	(kg/*l*)
N_{max}	18	25	28	(mg/kg)

TABLE 5-8

Model Results for Lead
Contoocook River, New Hampshire

Parameter	Cell 1	Cell 2	Cell 3	(Units)
k_d	234	234	234	(*l*/kg)
f_d	0.99	0.99	0.99	(unitless)
k_l	0.0	0.0	0.0	(m/sec)
k_g	0.0	0.0	0.0	(m/sec)
k	0.0	0.0	0.0	(m/sec)
k_b	0.0	0.0	0.0	(sec^{-1})
k_{agg}	0.0	0.0	0.0	(sec^{-1})
K_{tot}	0.0	0.0	0.0	(sec^{-1})
γ	1.0	1.0	1.0	(unitless)
Γ	1.0	1.0	1.0	(unitless)
C/W_0	8.0×10^{-2}	5.9×10^{-2}	5.6×10^{-2}	(sec/*l*)
C_d/W_0	7.9×10^{-3}	5.8×10^{-2}	5.5×10^{-2}	(sec/*l*)
W_0/N	7.3×10^{-3}	7.3×10^{-3}	7.3×10^{-3}	(kg/sec)
SRR_{SW}	5.7×10^{-4}	4.2×10^{-4}	4.0×10^{-4}	(kg/*l*)
N_{max}	9	12	12	(mg/kg)

k_d = 234 l/kg, an appropriate value for sand (U.S. EPA, 1986a) within the range reported by Baes, et al. (1984). From Equation 5-14 with $r_{sl} = 5\times10^{-5}$ kg/l, the fraction of lead dissolved is:

$$f_d = \frac{1}{1 + k_d\, r_{sl}}$$

$$= \frac{1}{1 + (234)(5\times10^{-5})}$$

$$= 0.99$$

This result indicates that lead will be distributed throughout the water column in dissolved (99%) and particulate (1%) form.

5.6.1.1.3. Estimating Downstream Concentrations of Contaminant --The estimates of net contaminant loss rate (K_{tot}) derived in Section 5.6.1.1.2 can be combined with values for the top area of each cell (A) and the flow rate for each cell (Q_f) to derive estimates of γ, Γ, and ultimately C_n/W_0. Values of the dimensionless transport factors, γ, and the cumulative transport factor, Γ, are computed using Equations 5-5 and 5-6, respectively: results are summarized in Table 5-7 for benzene and Table 5-8 for lead.

For benzene, Equation 5-5 gives:

$$\gamma_i = \frac{Q_{fi}}{Q_{fi} + A_i K_{toti}}$$

$$\gamma_1 = \frac{12.6}{12.6 + (515{,}000)(5.2\times10^{-6})} = 0.82$$

and

$$\gamma_2 = \frac{17.1}{17.1 + (88{,}200)(5.4\times10^{-6})} = 0.97$$

Since $\gamma_0 = 1$ by definition, it follows from Equation 5-6 that

$$\Gamma_3 = \prod_{i=0}^{2} \gamma_i = \gamma_0 \gamma_1 \gamma_2 = (1.00)(0.82)(0.97) = 0.80$$

The above result shows that approximately 80% of the benzene entering the surface water is transported downstream. Substituting Γ_3 into Equation 5-8 yields:

$$C_3/W_0 = \frac{\Gamma_3}{(Q_{f3} + A_3K_{tot3})}$$

$$= \frac{(0.80)}{18.0 + (232{,}000)(5.13\text{x}10^{-6})}$$

$$= 4.2\text{x}10^{-2} \quad (\text{sec}/l)$$

This concentration must be resolved into dissolved and suspended fractions, using the value of f_d calculated above from Equation 5-14:

$$C_{d3}/W_0 = f_d\,(C_3/W_0)$$

$$= (4.2\text{x}10^{-2}\ \text{sec}/l)(0.9976)$$

$$= 4.2\text{x}10^{-2}\ \text{sec}/l$$

This result can be interpreted to mean that each mg/sec of contaminant discharged from the aquifer to the Contoocook river is expected to increase the dissolved concentration of benzene at the location of the MEI by $4.2\text{x}10^{-2}$ mg/l.

Calculations are simpler for lead, since K_{tot} is assumed to equal zero. From Equations 5-5 and 5-6 it follows that γ=1 and Γ=1 for all cells in the cascade. Then from Equation 5-8,

$$C_3/W_0 = \frac{\Gamma_n}{(Q_{fn} + A_nK_{totn})}$$

$$= \frac{(1.0)}{[18.0 + (232{,}000)(0)]}$$

$$= 5.6\text{x}10^{-2}\ (\text{sec}/l)$$

The dissolved component of this concentration is then estimated from Equation 5-14:

$$\begin{aligned} C_{d3}/W_0 &= f_d\ (C_3/W_0) = (0.99)(5.6\text{x}10^{-2}) \\ &= 5.5\text{x}10^{-2}\ (\text{sec}/l) \end{aligned}$$

5.6.1.1.4. **Deriving Criteria** -- With Equations 5-9 and 5-10, the above results can be used to derive the maximum allowable dry-weight concentrations of contaminant in sludge allowed to accumulate in the Antrim facility. W_0/C_N for benzene was estimated for the Antrim facility in Section 4.7.1.1. It was estimated that for every mg/kg dry-weight of benzene in the sludge accumulating in lagoons at that site, approximately $1.0\text{x}10^{-3}$ mg/sec is expected to be discharged to the Contoocook river from the aquifer beneath the site, once the aquifer reaches steady state concentrations at the point of discharge. From Equation 5-9,

$$\begin{aligned} SRR_{SW} &= C_3/N = (C_3/W_0)(W_0/N) \\ &= (4.2\text{x}10^{-2}\ \text{sec}/l)(1.0\text{x}10^{-3}\ \text{kg/sec}) \\ &= 4.2\text{x}10^{-5}\ \text{kg}/l \end{aligned}$$

From Equation 5-10, SRR_{SW} can be applied to a RWC of $1.2\text{x}10^{-3}$ for benzene to derive the maximum allowable concentration of sludge to be allowed in the impoundment:

$$\begin{aligned} N_{max} &= RWC/SRR_{SW} \\ &= (1.2\text{x}10^{-3}\ \text{mg}/l)/(4.2\text{x}10^{-5}\ \text{kg}/l) \\ &= 28\ \text{mg/kg} \end{aligned}$$

Similarly, W_0/N was calculated to be 7.3×10^{-3} kg/l for lead in the Antrim lagoons. It follows that for lead,

$$\begin{aligned} SRR_{SW} &= C_3/N = (C_3/W_0)(W_0/N) \\ &= (5.5\times10^{-2})(7.3\times10^{-3}\ \text{kg/sec}) \\ &= 4.0\times10^{-4}\ \text{kg}/l \end{aligned}$$

and

$$\begin{aligned} N_{max} &= RWC/SRR_{SW} \\ &= (5.0\times10^{-3}\ \text{mg}/l)/(4.0\times10^{-4}\ \text{kg}/l) \\ &= 12\ \text{mg/kg} \end{aligned}$$

If dry-weight concentrations in the impoundment are greater than the above values then long-term storage should not be allowed in the facility.

For the exposure pathway involving human consumption of fish, the RWC for benzene can be calculated with methods described in Section 5.5.3:

$$RWC = (RL\ BW)/(q_1^*\ RE\ BCF\ I_f)$$

For an arbitrary choice of RL of 10^{-6} per lifetime, the RWC will be:

$$RWC = (10^{-6})(70)\ /\ [(0.029)(1.0)(5.2)(0.053)] = 8.8\times10^{-3}\ \text{mg}/l$$

Since fish could be caught anywhere in the river near the Antrim site, estimated dissolved concentrations in Cell 1 are used for the calculations. From Equation 5-8 and Table 5-7,

$$C_1/W_0 = \frac{\Gamma_n}{(Q_{f1} + A_1 K_{tot1})}$$

$$= \frac{(1)}{[12.6 + (515,000)(5.2\times10^{-6})]}$$

$$= 6.6\times10^{-2} \ (sec/l)$$

For comparison with RWC based on fish consumption, this concentration is not resolved into dissolved and suspended fractions. From Equation 5-9:

$$SRR_{sw} = C_1/N = (C_1/W_0)(W_0/N)$$

$$= (6.6\times10^{-3} \ sec/l)(1.0\times10^{-3} \ kg/sec)$$

$$= 6.6\times10^{-5} \ kg/l$$

From Equation 5-10:

$$N_{max} = RWC/SRR_{sw}$$

$$= (8.8\times10^{-3} \ mg/l)/(6.6\times10^{-5} \ kg/l)$$

$$= 130 \ mg/kg$$

For this particular site, the fish pathway of potential human exposure results in less stringent criteria for benzene than does the drinking water pathway. No bioconcentration factor is available for lead, so potential exposure through the fish consumption pathway is not considered.

5.6.1.2. SAMPLE CALCULATION: TULSA, OKLAHOMA -- The Northside Wastewater Treatment Plant is located at the confluence of Bird Creek and Mingo Creek in northeast Tulsa. Of five lagoons at the site, three are used for storage of sludge. The selected study reach extends for 20 km from the treatment plant to the City of Catoosa, Oklahoma. Catoosa was selected as the location of the MEI because it evidently is the closest community downstream from Tulsa's Northside Treatment Plant. After review of topographical and cross-sectional data for Bird Creek, the 20-km study corridor was modeled as the five-cell cascade described below:

- Cell 1 extends from the treatment plant boundary north along Mingo Creek for about 0.5 km to its confluence with Bird Creek;
- Cell 2 runs for 1.6 km along Bird Creek to the Elk Creek confluence;
- Cells 3, 4 and 5 cover about 18 kilometers on Bird Creek beginning at the Elm Creek confluence and ending at the Verdigras River confluence. The need for three cells along this reach was determined on the basis of significant changes on the hydraulic geometry of Bird Creek.

Mean annual flow values for these five cells were estimated from a regression of discharge on drainage area developed from USGS stream gages located in the Verdigras River watershed. Suspended sediment concentrations were obtained from field data reported by the U.S. Geological Survey, the U.S. Army Corps of Engineers and the U.S. Soil Conservation Service. Hydraulic geometry parameters were taken from HEC-2 input files prepared by the U.S. Army Corps of Engineers for use in Bird Creek flood studies. Input parameters for this site are summarized in Table 5-9, and represented schematically in Figure 5-6. The resulting estimates of contaminant concentrations and allowable mass loadings for benzene and lead are presented in Table 5-10 and 5-11. Methods used to compute the entries in those tables are identical to those used in the study of the Contoocook River, New Hampshire, as explained in detail in Section 5.6.1.1.

Results of those calculations show that benzene in sludge stored in the Tulsa facilities is not expected to cause adverse effects through the surface water pathways, but that criteria for lead (11 mg/kg) could be restrictive.

5.6.1.3. SAMPLE CALCULATION: PORTLAND, OREGON -- The City of Portland's municipal wastewater treatment plant and sludge disposal lagoon are located next to the Columbia Slough, which runs into the Willamette River approximately 7 kilometers downstream from the plant. The Columbia Slough, which bounds the treatment plant on the north, is considered a slackwater channel since significant flows are produced only as a result of storms. The confluence of the Columbia Slough with the Willamette River is approximately 1200 m upstream of the Columbia River. St. Helens, Oregon was chosen as the site for the MEI since it is the closest community downstream of the site. St. Helens

TABLE 5-9

Site-Specific Input Parameters
Bird Creek, Oklahoma

Parameter	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	(Units)
L	550	1,585	6,430	8,580	3,220	(m)
B	6	18	22	32	22	(m)
H	0.35	2.1	1.9	2.4	1.3	(m)
A	3,300	28,530	141,460	274,560	70,840	(m^2)
V	1,155	59,910	268,770	658,940	92,092	(m^3)
Q_f	0.28	16.7	17.0	17.5	17.5	(m^3/sec)
q_d	0.05	0.93	0.77	0.55	0.80	(m^2/sec)
U_s	0.14	0.44	0.41	0.23	0.62	(m/sec)
n	0.075	0.075	0.075	0.080	0.065	(unitless)
S	0.0004	0.0004	0.0004	0.0001	0.0012	(unitless)
L_m	123	--	--	--	--	(m)
U_{10}	4.9	4.9	4.9	4.9	4.9	(m/sec)
T	25	25	25	25	25	(°C)
r_{sl}	4×10^{-4}	4×10^{-4}	4×10^{-4}	4×10^{-4}	4×10^{-4}	(kg/*l*)
f_{oc}	0.1	0.1	0.1	0.1	0.1	(mg/*l*)

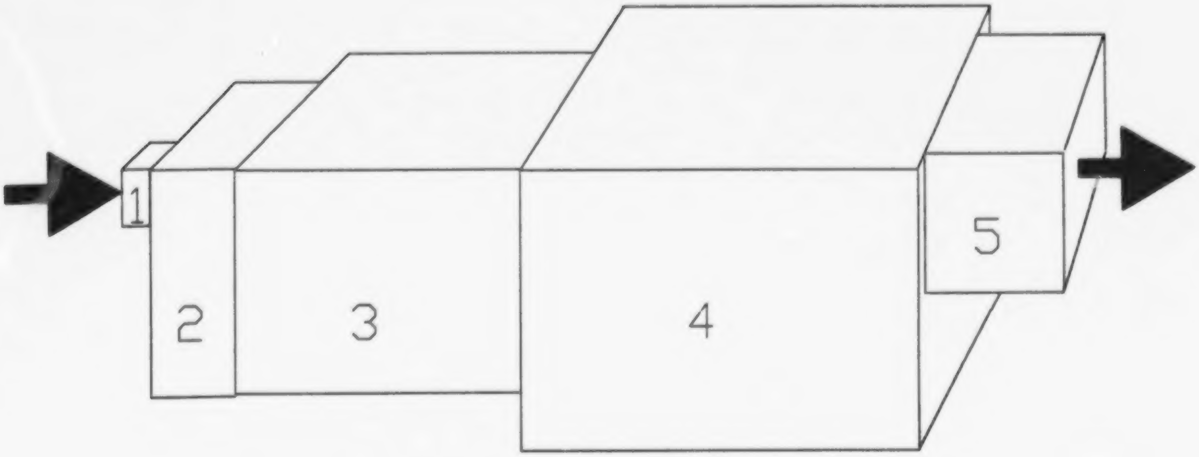


FIGURE 5-6

Five Cell Cascade for Bird Creek Near Tulsa, OK

TABLE 5-10

Model Results for Benzene
Bird Creek, Oklahoma

Parameter	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	(Units)
k_d	27	27	27	27	26.9	(l/kg)
f_d	0.9894	0.9894	0.9894	0.9894	0.9894	(unitless)
k_l	$6.9\text{x}10^{-5}$	$6.9\text{x}10^{-5}$	$6.9\text{x}10^{-5}$	$6.9\text{x}10^{-5}$	$6.89\text{x}10^{-5}$	(m/sec)
k_g	$2.7\text{x}10^{-3}$	$2.4\text{x}10^{-3}$	$2.2\text{x}10^{-3}$	$2.2\text{x}10^{-3}$	$2.3\text{x}10^{-3}$	(m/sec)
k	$6.2\text{x}10^{-5}$	$6.1\text{x}10^{-5}$	$6.1\text{x}10^{-5}$	$6.0\text{x}10^{-5}$	$6.1\text{x}10^{-5}$	(m/sec)
k_{agg}	$1.7\text{x}10^{-6}$	$1.7\text{x}10^{-6}$	$1.7\text{x}10^{-6}$	$1.7\text{x}10^{-6}$	$1.70\text{x}10^{-6}$	(sec^{-1})
K_{tot}	$6.2\text{x}10^{-5}$	$6.4\text{x}10^{-5}$	$6.3\text{x}10^{-5}$	$6.4\text{x}10^{-5}$	$6.2\text{x}10^{-5}$	(m/sec)
Q_f	0.28	16.7	17.0	17.5	17.5	(m^3/sec)
γ	0.58	0.90	0.66	0.50	0.80	(unitless)
Γ	1.0	0.58	0.52	0.34	0.17	(unitless)
C/W_0	2.1	$3.1\text{x}10^{-2}$	$2.0\text{x}10^{-2}$	$9.8\text{x}10^{-3}$	$7.8\text{x}10^{-3}$	(sec/l)
C_d/W_0	2.0	$3.1\text{x}10^{-2}$	$2.0\text{x}10^{-2}$	$9.7\text{x}10^{-3}$	$7.7\text{x}10^{-3}$	(sec/l)
W_0/N	$6.5\text{x}10^{-9}$	$6.5\text{x}10^{-9}$	$6.5\text{x}10^{-9}$	$6.5\text{x}10^{-9}$	$6.5\text{x}10^{-9}$	(kg/sec)
SRR_{SW}	$1.3\text{x}10^{-8}$	$2.0\text{x}10^{-10}$	$1.3\text{x}10^{-10}$	$6.3\text{x}10^{-11}$	$5.0\text{x}10^{-11}$	(kg/l)
N_{max}	9000	$> 1\text{x}10^{6}$	$> 1\text{x}10^{6}$	$> 1\text{x}10^{6}$	$> 1\text{x}10^{6}$	(mg/kg)

TABLE 5-11

Model Results for Lead
Bird Creek, Oklahoma

Parameter	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	(Units)
k_d	234	234	234	234	234	(l/kg)
f_d	0.91	0.91	0.91	0.91	0.91	(unitless)
k_l	0.0	0.0	0.0	0.0	0.0	(m/sec)
k_g	0.0	0.0	0.0	0.0	0.0	(m/sec)
k	0.0	0.0	0.0	0.0	0.0	(m/sec)
k_{agg}	0.0	0.0	0.0	0.0	0.0	(sec^{-1})
K_{tot}	0.0	0.0	0.0	0.0	0.0	(m/sec)
γ	1.00	1.00	1.00	1.00	1.00	(unitless)
Γ	1.00	1.00	1.00	1.00	1.00	(unitless)
C/W_0	3.6	0.060	0.059	0.057	0.057	(sec/l)
C_d/W_0	3.3	0.055	0.054	0.052	0.052	(sec/l)
W_0/N	$8.3x10^{-3}$	$8.3x10^{-3}$	$8.3x10^{-3}$	$8.3x10^{-3}$	$8.3x10^{-3}$	(kg/sec)
SRR_{sw}	$2.7x10^{-2}$	$4.6x10^{-4}$	$4.5x10^{-4}$	$4.4x10^{-4}$	$4.4x10^{-5}$	(kg/l)
N_{max}	0.18	11	11	11	11	(mg/kg)

is located approximately 24.5 km downstream of from the confluence of the Willamette and the Columbia rivers.

Two unique factors influenced the formation of the conceptual model: (1) the fact that the Columbia Slough is a slackwater channel which receives the majority of inflow from agricultural drainage and (2) the fact that the Columbia River is influenced by the tidal action of the Pacific Ocean. These factors were considered in specifying the following 5-cell model, which is depicted in

Figure 5-7:

- Cell 1 represents the Columbia Slough and is assumed to have an extremely low flow rate with a high detention time.
- Cell 2 covers the 1200 m section of the Willamette River.
- Cells 3, 4 and 5 represent different characteristics of the Columbia River. Three segments are required for adequate representation due to the significant changes in channel geometry and tributary inflow of the Lewis River at approximately 1.6 km above the location of the MEI.

Most hydraulic data were taken from HEC-2 input files prepared by the U.S. Army Corp of Engineers. The mean annual flow values were estimates established by the U.S. Geological Survey. Site-specific input parameters are summarized in Table 5-12; the resulting estimates of contaminant concentrations and allowable mass loadings for benzene and lead are presented in Tables 5-13 and 5-14, respectively. As can be seen from these tables, if Cell 5 is taken as the point of compliance, then dry-weight concentrations of benzene in sludge will be restricted to 1000 mg/kg, and concentrations of lead will be restricted to 160 mg/kg.

Human exposure through the drinking water pathway from the Portland site is limited by significant dilution of benzene within the Williamette and Columbia rivers prior to ingestion by humans. For the fish ingestion pathway, however, the first modeled cell of the Columbia Slough is taken as the point of compliance. As can be seen from Table 5-13, the source-receptor ratio for benzene in Cell #1 is 3.7×10^{-3}. This value, derived for the drinking water pathway, does not include contaminant adsorbed to suspended sediment; for the fish ingestion pathway, SRR_{su} is approximately 3.8×10^{-3}. Then N_{max} for this

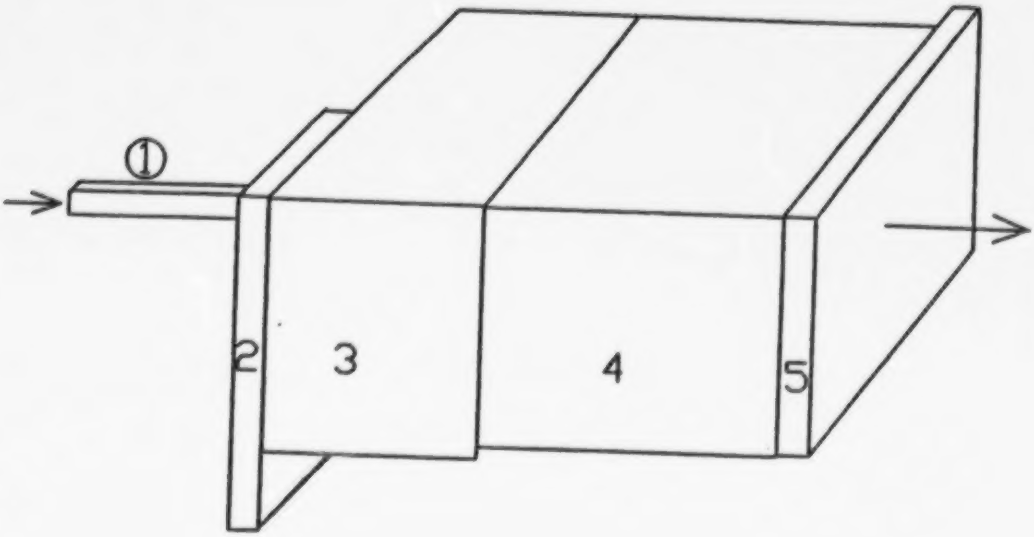


FIGURE 5-7

Five Cell Cascade for Columbia Slough Near Portland, OR

TABLE 5-12

Site-Specific Input Parameters
Columbia Slough, Oregon

Parameter	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	(Units)
L	6,980	1,220	9,980	12,875	1,610	(m)
B	55	445	825	830	955	(m)
H	1.1	11.3	10.1	10.2	10.3	(m)
A	383,900	542,900	8,232,500	10,686,250	1,537,550	(m^2)
V	575,850	7,546,000	88,100,000	111,642,500	16,020,000	(m^3)
Q_f	2.3	838	5,660	5,660	5,776	(m^3/sec)
q_d	0.04	1.88	6.86	6.82	6.05	(m^2/sec)
U_s	0.36	0.17	0.68	0.67	0.59	(m/sec)
n	0.030	0.030	0.024	0.029	0.030	(unitless)
S	1×10^{-6}	1×10^{-6}	1.2×10^{-5}	1.7×10^{-5}	1.4×10^{-5}	(unitless)
L_m	8,445	--	--	--	--	(m)
U_{10}	3.6	3.6	3.6	3.6	3.6	(m/sec)
T	25	25	25	25	25	(°C)
r_{sl}	1×10^{-4}	1×10^{-4}	1×10^{-4}	1×10^{-4}	1×10^{-4}	(kg/*l*)
f_{oc}	0.2	0.2	0.2	0.2	0.2	(unitless)

TABLE 5-13

Model Results for Benzene
Columbia Slough, Oregon

Parameter	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	(Units)
k_d	54	54	54	54	54	(l/kg)
f_d	0.995	0.995	0.995	0.995	0.995	(unitless)
k_l	3.7×10^{-5}	3.7×10^{-5}	3.7×10^{-5}	3.7×10^{-5}	3.7×10^{-5}	(m/sec)
k_g	1.7×10^{-3}	1.6×10^{-3}	1.4×10^{-3}	1.4×10^{-3}	1.5×10^{-3}	(m/sec)
k	3.4×10^{-5}	3.4×10^{-5}	3.3×10^{-5}	3.3×10^{-5}	3.4×10^{-5}	(m/sec)
k_{agg}	1.70×10^{-6}	1.70×10^{-6}	1.70×10^{-6}	1.70×10^{-6}	1.70×10^{-6}	(sec^{-1})
K_{tot}	3.6×10^{-5}	5.3×10^{-5}	5.0×10^{-5}	5.0×10^{-5}	5.1×10^{-5}	(m/sec)
γ	0.14	0.97	0.93	0.91	0.99	(unitless)
Γ	1.00	0.14	0.14	0.13	0.12	(unitless)
C/W_0	6.3×10^{-2}	1.7×10^{-4}	2.3×10^{-5}	2.1×10^{-5}	2.0×10^{-5}	(sec/l)
C_d/W_0	6.2×10^{-2}	1.7×10^{-4}	2.3×10^{-5}	2.1×10^{-5}	2.0×10^{-5}	(sec/l)
W_0/N	0.059	0.059	0.059	0.059	0.059	(kg/sec)
SRR_{SW}	3.7×10^{-3}	9.8×10^{-6}	1.4×10^{-6}	1.2×10^{-6}	1.2×10^{-6}	(kg/l)
N_{max}	0.33	120	890	970	1000	(mg/kg)

TABLE 5-14

Model Results for Lead
Columbia Slough, Oregon

Parameter	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	(Units)
k_d	234	234	234	234	234	(l/kg)
f_d	0.98	0.98	0.98	0.98	0.98	(unitless)
k_l	0.00	0.00	0.00	0.00	0.00	(m/sec)
k_g	0.00	0.00	0.00	0.00	0.00	(m/sec)
k	0.00	0.00	0.00	0.00	0.00	(m/sec)
k_{agg}	0.00	0.00	0.00	0.00	0.00	(sec^{-1})
K_{tot}	0.00	0.00	0.00	0.00	0.00	(m/sec)
γ	1.00	1.00	1.00	1.00	1.00	(unitless)
Γ	1.00	1.00	1.00	1.00	1.00	(unitless)
C/W_0	$4.3x10^{-1}$	$1.2x10^{-3}$	$1.8x10^{-4}$	$1.8x10^{-4}$	$1.7x10^{-4}$	(sec/l)
C_d/W_0	$4.2x10^{-1}$	$1.2x10^{-3}$	$1.7x10^{-4}$	$1.7x10^{-4}$	$1.7x10^{-4}$	(sec/l)
W_0/N	0.19	0.19	0.19	0.19	0.19	(kg/sec)
SRR_{sw}	0.08	$2.2x10^{-4}$	$3.3x10^{-5}$	$3.3x10^{-5}$	$3.2x10^{-5}$	(kg/l)
N_{max}	0.06	23	154	154	157	(mg/kg)

pathway can be calculated from Equation 5-10 for RWC = 8.8×10^{-3} (as derived in Section 5.6.1.1):

$$N_{max} = RWC/SRR_{SU} = (8.8\times10^{-3}\ mg/l)\ /\ (3.8\times10^{-3}\ kg/l) = 2.3\ mg/kg$$

Because this value is based on expected concentrations in Cell #1 of the system, it is significantly more restrictive than criteria based on the drinking water pathway.

5.6.2. Analysis of Exposure for the Most Exposed Populations. Tier 2 calculations provide for the MEI an upper-bound estimate of maximum concentrations likely to be encountered at a specified receptor location downstream from the point of groundwater discharge into the stream. They also provide a derivation of maximum allowable sludge concentrations, based on reference water concentrations, site characteristics and a specified receptor distance.

The distribution of potential exposure over the entire exposed population may also be of interest. For the sample calculations presented here, the most exposed population (MEP) to be considered will be those individuals who regularly fish in the surface water body at the point of contaminated groundwater discharge.

5.6.2.1. MEP ANALYSIS: ANTRIM, NEW HAMPSHIRE -- To perform an analysis of the most exposed population (MEP) for the fish ingestion pathway, two pieces of information are needed: a distribution of fish consumption in the exposed population, and the number of persons exposed to fish caught in the contaminated water. As discussed in Section 5.5.2.2., U.S. EPA (1980a) reports data for fish consumption in the United States by a number of demographic variables. These data were summarized in Table 5-4, and included consumption of fish from all sources (freshwater and marine). This table provided the mean and upper 95th percentile of fish consumption for New England (for the New Hampshire site analysis), but were based on data analyzed in 1980. From the table, it can be seen that average fish consumption for the United States was estimated to be about 14.4 g/day. As discussed in Section 5.5.2.2, more recent data (adjusted for recreational catch) suggest an average consumption rate of 21.3 grams, about 48% higher than values reported in Table 5-4.

If it is assumed that the distribution of fish consumption within each population is approximately log-normal, the values in Table 5-4 can be used to derive a geometric mean and geometric standard deviation of fish consumption for each region in the U.S. For New Hampshire, the appropriate distribution is described for the New England population, which is reported to have a mean consumption rate of 16.3 g/day, and a 95th percentile value of 46.5 g/day. These values suggest a geometric mean of 13.3 g/day and geometric standard deviation of 1.89. If that mean is adjusted by 46% to correspond to the 1985 average value from Table 5-3, and if the geometric standard deviation is held constant, then the revised distribution has a geometric mean of 19.7 and arithmetic mean of 24.1. It will be assumed that 50% of the fish ingested is from freshwater sources, which yields an estimated geometric mean of 9.8 g/day or arithmetic mean of 12.1 g/day of freshwater fish consumed by the average resident of New England.

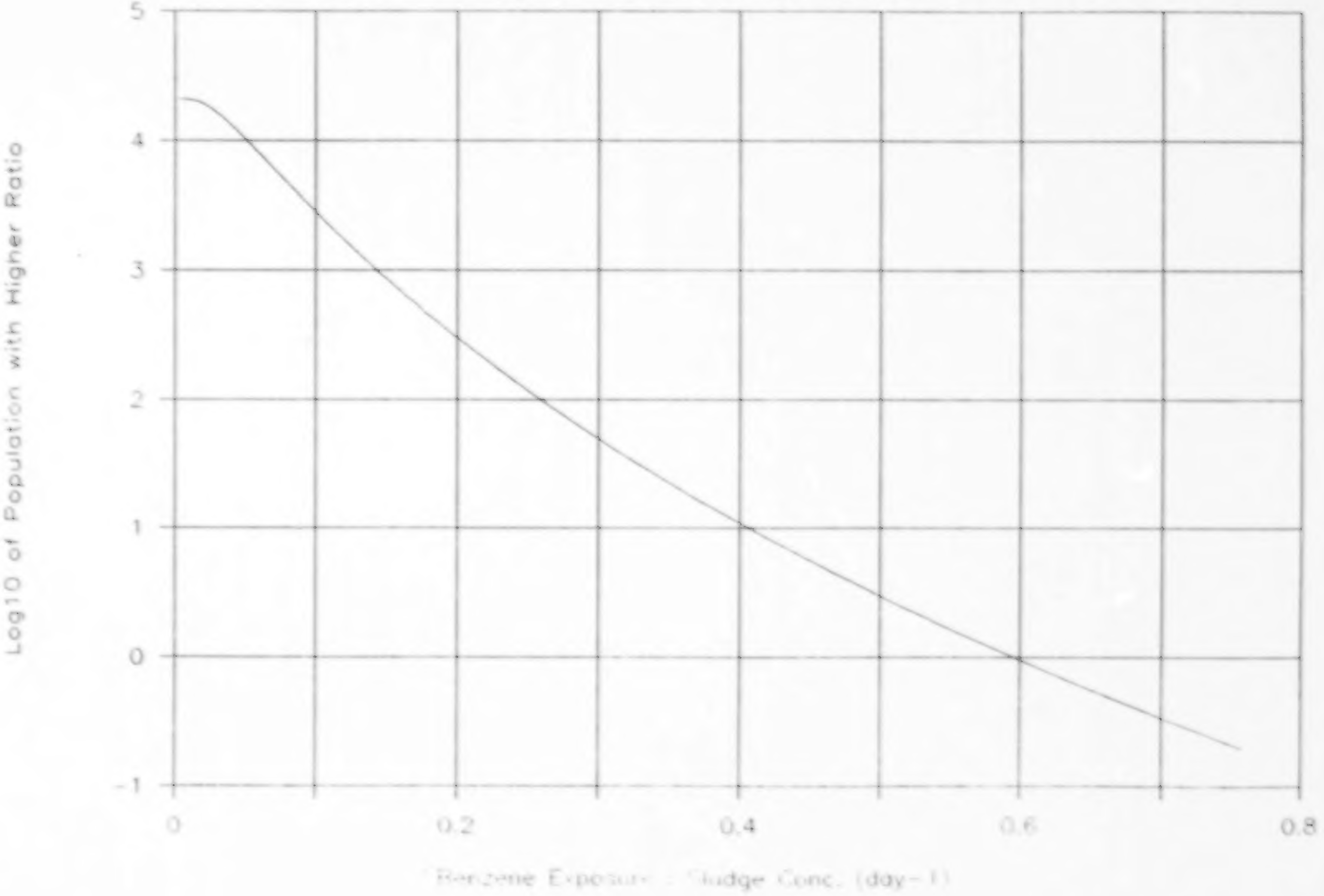
Next, the size of the exposed population must be determined. It will be assumed that fishing behavior is uniformly distributed (by flow) over the freshwater resources in New Hampshire. Estimated flow in the Contoocook river near the Antrim site is about 12.6 m^3/sec, or about 2% of the total freshwater budget of New Hampshire (about 630 m^3/sec). If freshwater fish consumed within these waters are consumed by 2% of the New Hampshire population, then about (978,000)(0.02)=20,000 persons are potentially exposed.

The estimated bioconcentration factor for benzene in fish is about 5.2 *l*/kg. If this estimate is combined with the source receptor ratio $SRR_{su} = 6.8 \times 10^{-5}$ kg/*l* estimated for Cell #1 of the Antrim site, then a distribution can be derived for the ratio of exposure (in mg/kg-day) to dry-weight concentration of benzene in sludge stored by the facility (in mg/kg). Combining this distribution with the estimated size of the exposed population yields the results presented in Figure 5-8. As can be seen from the figure, the estimated ratio of potential exposure (in mg/kg-day) to sludge concentration (in mg/kg dry-weight) is greater than 0.15 (day^{-1}) for about 1000 persons. For about 100 persons, the ratio is

FIGURE 5-8

Ratio of Exposure to Sludge Conc.

By Size of MEP Near Antrim, NH



greater than 2.5. According to the figure, the ratio of exposure to sludge concentration for the MEI would be about 0.6 day^{-1}.

5.6.2.2. SAMPLE CALCULATION: TULSA, OKLAHOMA -- Calculations for the Tulsa site are similar to those just described for Antrim. Table 5-7 showed that residents of the West South Central region consumed 14.4 g/day, with an upper 95th percentile of 43.6 g/day. If the distribution is log-normal, then these estimates suggest a geometric mean of 11.0 and geometric standard deviation of 2.1. This mean is adjusted by 1.48 to correspond to estimates based on 1985 data, and is multiplied by 0.5 to include only freshwater fish. The resulting geometric mean is (11.0)(1.48)(0.5) = 8.1 g/day, corresponding to an arithmetic mean of 12.1.

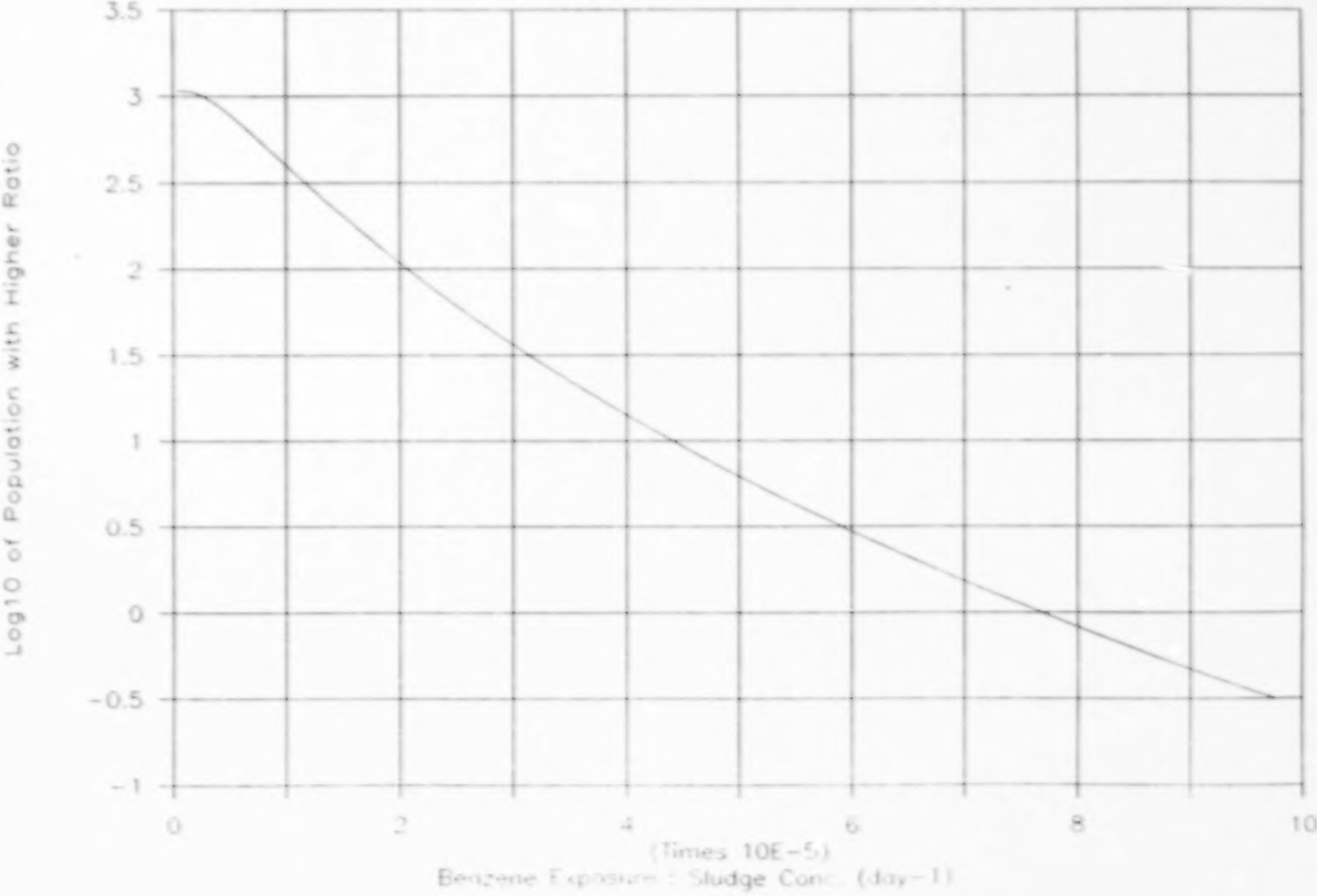
The estimated rate of flow for the river system considered for Tulsa was about 0.28 m^3/sec. This flow represents about 0.03% of the total freshwater budget of Oklahoma (about 900 m^3/sec), so it is assumed that about 0.03% of the total population of Oklahoma (or about 1000 persons) consumes fish caught from the study area. This estimate is combined with the estimated distribution of fish consumption described above, with a bioconcentration factor of 5.2, and with the estimated SRR$_{SW}$ for Cell #1 of this site of 6.5x10^{-9} kg/ℓ to yield the distribution graphed in Figure 5-9. As can be seen from the figure, the ratios of predicted exposure to sludge concentration of benzene are much lower for Tulsa than were estimated for Antrim. Of the 1,000 persons exposed, about 100 can expect exposure to more than 2.5x10^{-5} mg/kg-day of benzene for each mg/kg of benzene in sludge stored at the site. The ratio could reach as high as 8x10^{-5} for the MEI. As discussed in Section 5.6.1.2, this low exposure is explained by the slow water flow (and greater opportunity for contaminant decay) within the aquifer beneath the Tulsa site.

5.6.2.3. SAMPLE CALCULATION: PORTLAND, OREGON -- The same calculations can be repeated for the Portland, Oregon site to derive an estimated distribution of exposure to Oregon residents as a function of the concentration of benzene in sludge deposited in the impoundment near the Columbia Boulevard plant. For the Pacific region, Table 5-4 reported that the average rate of consumption of fish was 14.2 g/day, with a 95th percentile value of 39.2 g/day. If the distribution is log-normal, then it can be

FIGURE 5-9

Ratio of Exposure to Sludge Conc.

By Size of MEP Near Tulsa, OK



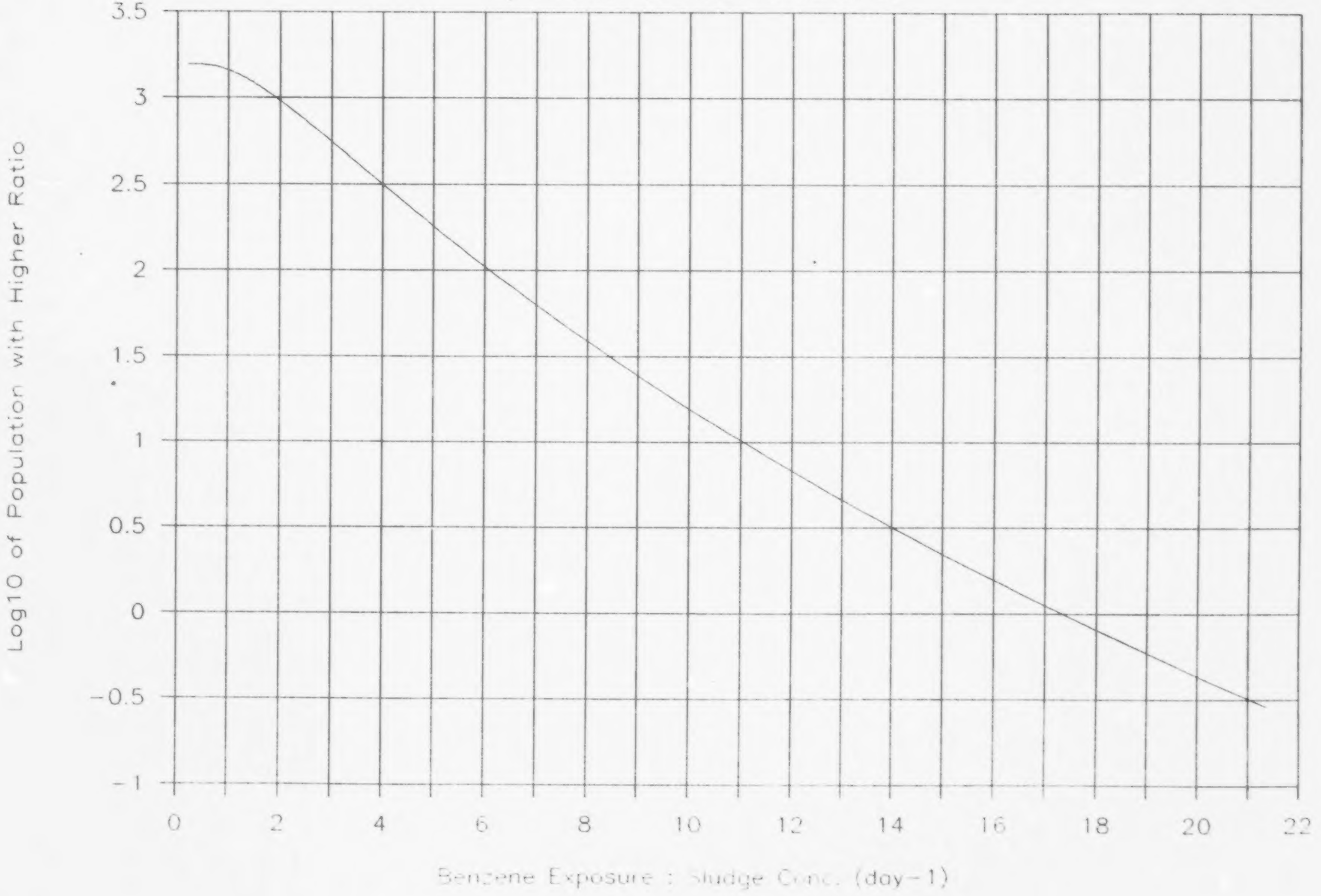
described by a geometric mean of 11.8 and geometric standard deviation of 1.9 g/day. These values are adjusted as described above to yield a revised geometric mean of (11.8)(1.48)(0.5) = 8.7 g/day, corresponding to an arithmetic mean of 10.5 g/day.

The flow of the Columbia Slough is estimated to be about 2.3 m^3/sec, or about 0.06% of the estimated 4000 m^3/sec total freshwater budget for Oregon. If 0.06% of the total population of Oregon is potentially exposed, then the size of the exposed population is about 1600. This estimate is combined with the distribution derived above, and with a source-receptor ratio of $SRR_{sw} = 3.7 \times 10^{-3}$ for Cell #1 in the Columbia Slough to yield the distribution described by Figure 5-10. As can be seen from the figure, about 100 persons are exposed to a ratio higher than 6, and the MEI is exposed to about 17 mg/kg-day for every mg/kg of dry-weight concentration of benzene in sludge received by the impoundment. Because for the relatively high source-receptor ratio for Cell #1 in this system, estimated exposures through the fish consumption pathway are considerably higher for the Portland site than for Antrim or Tulsa.

FIGURE 5-10

Ratio of Exposure to Sludge Conc.

By Size of MEP Near Portland, OR



6. DERIVATION OF CRITERIA FOR THE AIR PATHWAY

6.1. OVERVIEW OF THE METHOD

This chapter describes a methodology for deriving criteria for the air pathway of exposure from surface disposal facilities. The approach is based on the assessment of risks to human health and is suitable for both site-specific and national application. As with the exposure pathways discussed in Chapters 4 and 5, the methodology for deriving criteria based on volatilization involves a two-tiered approach; both tiers seek to derive criteria sufficiently protective for the most exposed individual (MEI). The first tier involves the use of a mathematical model to describe the migration of sludge contaminants from liquid within a lagoon to air above the lagoon surface, and the transport of contaminated air to downwind receptor locations. Based on this model, Tier 1 calculations prepare national, numerical sludge criteria based on a reasonable worst case scenario. The scenario is chosen such that conditions at actual facilities are unlikely to result in higher potential for human or non-human exposure than conditions assumed for the derivation of Tier 1 criteria. Tier 2 calculations derive limits to contaminant concentrations in sludge based on the same mathematical model but with site-specific input values. The property boundary may be selected as the point of compliance, since residential dwellings constructed near the property boundary could be affected by contaminated air, with subsequent public health implications. Alternatively, the nearest actual downwind residential location could be analyzed.

Both tiers seek to limit concentrations of volatile contaminants in sludge so that they will not lead to air concentrations that exceed health-based limits. The first derives maximum allowable contaminant concentrations in ambient air, based on health effects data for each contaminant under consideration. The second determines expected rates of volatile emissions from the lagoon, as a function of contaminant concentrations in sludge received by or accumulating in the impoundment. This step uses a two-layer resistance model to calculate volatilization as a function of expected contaminant concentrations within the impoundment; these concentrations are determined with a mass balance approach. The third step predicts expected average contaminant concentrations in ambient air at the specified location of the MEI, as a function of expected volatile emissions. Calculations are based on a mathematical model of contaminant dispersion during wind transport. The fourth step combines results from the other three, to derive maximum allowable concentrations in sludge. Each of these steps will be discussed in greater detail in Sections 6.3 and 6.5.

Chapters 4 and 5 described methods for deriving criteria related to potential exposure through groundwater and surface water pathways; for the calculations described in those chapters, no effort was made to discriminate between different kinds of active impoundments. For estimating potential exposure to contaminants volatilizing from surface disposal facilities, however, it is useful to classify lagoons into three categories: (1) well-mixed lagoons with continuous inflow, (2) poorly mixed lagoons with continuous inflow, and (3) impoundments receiving occasional or periodic deposits of sludge. As will be discussed below, differences in conditions at these types of lagoons suggest different choices of mathematical models for estimating volatile emissions.

6.2. ASSUMPTIONS

Table 6-1 summarizes some key assumptions used in the methodology described below. These assumptions can be grouped into two categories: those involved in the estimation of emissions from surface disposal facilities, and those involved in the estimation of the transport of those emitted contaminants by wind. Volatilization is predicted with a mass balance approach in which biodegradation, volatilization, settling to the sludge layer, seepage, and removal through effluent are the only significant loss processes for contaminant. It is assumed that adsorbed contaminant within the sludge layer does not contribute to volatilization, but that adsorbed contaminant on suspended solids maintains an equilibrium with the dissolved concentration in the liquid layer, and acts as a source of contaminant as contaminant volatilizes from dissolved phase. That liquid layer is assumed to cover the entire facility, and to maintain a constant volume throughout the lifetime of the lagoon. It is assumed that biological degradation within the liquid layer is first order. Estimation of the extent to which concentrations of contaminant are attenuated during wind transport to a receptor location is based on a Gaussian plume dispersion model that assumes meteorologic conditions are constant between the emissions source and the receptor. It is further assumed that atmospheric losses of the contaminant can be described as a first order process. These and other assumptions will be discussed in more detail throughout the remainder of this chapter.

TABLE 6-1

Assumptions for Methodology to Analyze the Air Pathway

Functional Area	Assumption	Ramifications
Emissions from Impoundment	Biodegradation, volatilization, seepage, and removal in effluent are the only significant loss processes for contaminant.	Underpredicts emissions if photolysis, hydrolysis, or other loss processes are significant.
	Plug flow scenario assumes one-dimensional flow of water and contaminant.	Unknown.
	Equilibrium between dissolved and adsorbed concentrations of contaminant in suspended solids within the impoundment.	Underpredicts emissions to the extent that bottom layer acts as a contaminant reservoir for release through volatilization.
	Adsorbed contaminant in sludge layer does not contribute to volatilization.	Underpredicts emissions.
	Rate of volatile emissions of emissions from impoundment can be described by two-phase resistance model.	Unknown.
	Average temperature of 25° is assumed for all impoundments.	Unknown.
	Entire surface of impoundment is assumed to be in liquid phase.	Unknown in circumstances where accumulating bottom layer of sludge solids reaches the impoundment surface. Overpredicts emissions from frozen impoundments.
	Concentration of suspended solids is uniform within the liquid layer and throughout the active lifetime of the facility.	Unknown.
	Use of diffused air in a wastewater treatment impoundment does not affect solids content in surface layer of liquid.	Underpredicts emissions if diffused air increases concentrations of suspended solids near the impoundment surface.
	For facilities with continuous flow, inflow of liquid is assumed to equal outflow (including seepage).	Unknown.

TABLE 6-1 (cont.)

Functional Area	Assumption	Ramifications
	Biological degradation is first order.	Will overpredict degradation (and underpredict emissions) at high concentrations of contaminant.
	The rate of contaminant loss through effluent and seepage can be approximated by QC_l for well mixed facilities, and by $QC_l(L)$ for poorly mixed facilities.	Unknown.
Atmospheric Transport	Atmospheric conditions are constant between the site and the receptor location.	Unknown.
	Receptor is at ground level.	Unknown.
	Emission rates for a particular site are constant for each contaminant.	May underpredict acute exposure.
	Contaminant plume follows a Gaussian distribution.	Unknown.
	Atmospheric losses of contaminant are first-order.	Unknown.

6.3. CALCULATIONS

Calculations for Tier 1 and Tier 2 are based on the assumption of mass balance for contaminants entering the lagoon. The methodology assumes that the quantity of each contaminant entering the lagoon in a specified time interval is equal to the sum of the quantity removed or destroyed, plus the quantity retained:

$$M_I = M_O + \Delta M_T \tag{6-1}$$

where:

M_I = mass of contaminant entering the lagoon (g/sec)
M_O = mass of contaminant degrading or leaving the lagoon (g/sec)
ΔM_T = change in total contaminant mass stored within the lagoon (g/sec)

Contaminants enter the impoundment through periodic or occasional deposits of sludge, or through continuous inflow of sludge or wastewater. They leave through discharged effluent, seepage beneath the lagoon in dissolved phase, biodegradation and volatilization. Other potential routes by which contaminants might be eliminated include photodecomposition, hydrolysis, oxidation/reduction and hydroxyl radical reactions. For surface impoundments and wastewater treatment lagoons, however, these other pathways are assumed to be of either negligible or secondary importance (U.S. EPA, 1987a). Equation 6-1 can thus be expanded to include those removal processes to be considered:

$$M_I = M_E + M_B + M_{VD} + M_{VA} + \Delta M_T \tag{6-2}$$

where:

M_E = the mass of contaminant removed with effluent, or with seepage from the bottom of the impoundment (g/sec)
M_B = the mass of contaminant removed by biodegradation (g/sec)
M_{VD} = the mass of contaminant removed by volatilization that results from diffused air (g/sec) and
M_{VA} = the mass of contaminant removed by volatilization that results from wind blowing across the impoundment (g/sec)

Each of the components of Equation 6-2 will now be discussed in more detail.

6.3.1. Influent and Effluent Flow. The rate at which contaminants enter a sludge disposal facility depends on the type of facility involved. The present methodology considers facilities in which sludge is deposited continuously, and those in which sludge is deposited periodically or occasionally. For impoundments for which inflow of sludge or wastewater is continuous, the mass of contaminant entering the lagoon per unit of time is described by:

$$M_I = Q\, C_0$$

where:

M_I = mass of contaminant entering the lagoon (g/sec)
Q = the liquid volumetric flow rate (m^3/sec)
C_0 = the concentration of the contaminant (both dissolved and adsorbed) in liquid at the inlet (mg/l or g/m^3)

Similarly, the rate of contaminant loss associated with liquid flow out of the system is given by the relation:

$$M_E = Q\, C_E \qquad (6\text{-}3)$$

where:

M_E = mass of contaminant removed from the lagoon through effluent or seepage (g/sec)
Q = the volumetric flow of liquid leaving the impoundment as effluent or seepage (m^3/sec)
C_E = the concentration of the contaminant in effluent or seepage (g/m^3)

For surface impoundment sites with continuous influent flow, liquid flow rates into and out of the lagoon will be assumed to be equal (i.e., net precipitation will be ignored). This assumption results in a constant-volume model for these systems. It should also be noted that contaminant concentrations in effluent and seepage are here assumed to be identical. This assumption will be further discussed below.

For surface impoundments receiving periodic deposits of sludge, the mass of contaminant in the facility decreases throughout the time interval between sludge deposits. Between deposits, contaminant is lost to seepage, effluent, biodegradation, volatilization and other loss processes. For some impoundments, sludge may be removed between deposits, or deposits may be sufficiently infrequent that yearly average rates of emissions can be calculated based on time-averaged estimates of emissions from a single deposit of sludge. For these facilities, the mass of contaminant received in a single deposit can be described as:

$$M_I = V_{dep} C_0$$

where V_{dep} is the average volume of sludge received in a single deposit.

6.3.2. Contaminant Mass Lost to Biodegradation. The rate at which contaminant mass is lost through biological degradation is approximated as a first-order process. For very high in-liquid concentrations, the assumption of first-order biodegradation will tend to overestimate biodegradation rates, and hence underestimate rates of contaminant volatilization. A first-order model, however, can provide a reasonable approximation of biodegradation rates at low contaminant concentrations. A more accurate relation valid for both high and low concentrations is described by Monod-type kinetics (U.S. EPA, 1987a). The assumption of Monod-type kinetics, however, would result in a nonlinear mass balance equation that would require numerical solution, and would require a more complex model for volatile emissions than the one proposed here. For liquid concentrations of contaminant lower than the Monod half-saturation constant, biodegradation can be approximated with reasonable accuracy as a first-order process described by:

$$M_B = k_b V C_l$$

where:

M_B = the loss rate of contaminant mass through biodegradation (g/sec)
k_b = the first-order biodegradation rate constant (sec^{-1})
V = the volume of the liquid considered (m^3)
C_l = the concentration of the contaminant in the liquid (g/m^3)

Biodegradation rates for organic contaminants are available from the literature, and can be adjusted for different temperatures with Equation 5-27 from Chapter 5.

Biological activity might also generate mass of some toxic chemicals within the lagoon; for lack of sufficient data, however, this analysis assumes that all potential exposure is to contaminants contained in the sludge or wastewater as received by the impoundment.

6.3.3. Contaminant Mass Lost to Volatilization. The rate at which the contaminant volatilizes to air is described by:

$$M_{VA} = k\,A\,C_l \tag{6-4}$$

where:

M_{VA} = the loss rate of contaminant mass through volatilization to ambient air (g/sec)
k = the overall mass transfer coefficient of the system (m/sec)
A = the area of the air-exposed surface of the lagoon (m^2)
C_l = the concentration of the contaminant in the liquid (g/m^3)

As in Chapter 5, the overall mass transfer coefficient (k) in Equation 6-4 is calculated based on a two-layer resistance model:

$$\frac{1}{k} = \frac{1}{k_l} + \frac{RT}{H_c k_g} \tag{6-5}$$

where:

k_l = the liquid phase mass transfer coefficient (m/sec)
R = the ideal gas constant = 8.21×10^{-5} (m^3-atm/mol-°K)
T = temperature (°K)
H_c = Henry's law constant for contaminant (m^3-atm/mol)
k_g = the gas phase mass transfer coefficient (m/sec)

Numerous methods for calculating k_l and k_g have been proposed in the literature. As in Chapter 5, this methodology follows U.S. EPA (1987a) and U.S. EPA (1989c) in selecting methods for calculating these coefficients; the methods are described in Chapter 5 by Equations 5-17 through 5-25.

6.3.4. Contaminant Mass Lost to Volatilization from Diffused Air. Volatilization via this pathway is applicable only to wastewater treatment facilities with diffused air systems. The contaminant mass loss rate from the system through diffused air is described by:

$$M_{VD} = Q_a \, (H_c/RT) \, C_l \qquad (6\text{-}6)$$

where:

M_{VD} = the emission rate of contaminant in diffused air (g/sec)
Q_a = the volumetric flow rate of diffused air through the system (m^3/sec)
H_c = Henry's law constant for the contaminant (atm-m^3/mol)
R = the ideal gas constant = 8.21×10^{-5} (atm-m^3/mol-°K)
T = temperature of the system (°K)
C_l = the concentration of the contaminant in the liquid (g/m^3)

Volatilization through diffused air contributes to total volatilization from the lagoon and must be included in estimates of potential exposure through the air pathway. Because this type of volatilization is of concern for wastewater treatment lagoons only, discussion of this pathway will be deferred until Section 6.3.7.4, where wastewater treatment lagoons will be considered. Estimation of volatilized emissions from diffused air will not be included in mass balance calculations for other types of facilities.

6.3.5. Change in Total Contaminant Mass Contained Within the Lagoon. The final term (ΔM_T) in Equations 6-1 and 6-2 represents the increase in contaminant mass stored within the lagoon that will result if the quantity of contaminant flowing into the impoundment exceeds the quantity removed or degraded. In most active lagoons, a bottom layer of sludge solids accumulates with time. Contaminant adsorbed to solids in this sludge layer represents one component of ΔM_T; the other component includes any increase over time in the mass of dissolved contaminant within the impoundment.

As will be discussed in Section 6.3.7.1, lagoons with continuous inflow and outflow of sludge or wastewater are modeled as if $\Delta M_T = 0$. The model for those lagoons ignores the possible accumulation of contaminant mass in the lagoon over time, and instead assumes that the steady-state concentrations of dissolved contaminant, adsorbed contaminant and total suspended solids all remain constant throughout the active lifetime of the lagoon. This assumption is accomplished by redefining Equation 6-2 to refer to a "control volume" within the lagoon, defined as the layer of liquid between the surface of the lagoon and the top of an idealized sludge layer on the impoundment's floor. Terms on the right side of Equation 6-2 then refer to contaminant removal from this control volume, and the definition of M_g is expanded to include the loss of adsorbed contaminant through settling of solids to the sludge layer at the bottom of the lagoon. Implicit in this re-definition is the assumption that adsorbed contaminant within the sludge layer does not contribute to concentrations in the upper layer of liquids, and so does not contribute to volatilization.

As redefined, M_g will not necessarily equal QC_l, as assumed in Equation 6-3. However, the methodology presented below will assume that QC_l provides a reasonable approximation of the total quantity of contaminant leaving the impoundment through effluent, seepage, or settling. To the extent that the increasing reserve of sludge mass and adsorbed contaminant increases dissolved concentrations of contaminant above the impoundment floor (and therefore increases rates of volatilization), the rates of volatile emissions predicted by this methodology may underestimate actual emissions.

Similarly, ΔM_T is assumed to equal zero for poorly mixed impoundments with continuous inflow. These impoundments, which are discussed in Section 6.3.7.2, are represented by a "plug flow" model, in which it is assumed that a differential flow element can be modeled without regard to accumulation of contaminant or sludge solids in the impoundment. As with well-mixed systems, this assumption is accomplished by assuming that the flow of liquid within the lagoon occurs within a control volume above the accumulating layer of sludge solids, and that the mass of contaminant that the model predicts will leave the far end of the lagoon per unit of time is comparable with the actual total of contaminant mass lost through effluent and seepage, plus the mass of contaminant lost from the control volume through settling. To the extent that this assumption is incorrect, the model may under- or overpredict actual emissions.

6.3.6. Mass of Contaminant Transferred from Adsorbed to Dissolved Phase.

Within the control volume, contaminant will exist both in dissolved phase and adsorbed to suspended solids; both phases must be considered for a meaningful estimate of volatile emissions. As volatilization removes dissolved contaminant from the impoundment, additional contaminant mass will be transferred from the suspended solids to dissolved phase. This transfer will not affect the overall mass balance described by Equation 6-2, but needs to be considered when estimating rates of volatilization.

For each type of facility discussed below, two separate mass balance equations are derived: one for the mass of a contaminant entering and leaving the liquid phase of the control volume and the other for the mass of a contaminant entering and leaving the (suspended) solid phase. In addition to various gain and loss terms, the equations contain a term describing the transfer of contaminant mass (positive or negative) from the solid to the liquid compartment of the system. This mass transfer will be denoted M_{sl} and will be conserved in the system: the gain by the liquid phase of the mass balance will equal the loss by the solid phase. As mentioned earlier, possible desorption of contaminant from the sludge layer on the floor of an impoundment will be ignored.

In this analysis, it is assumed that the concentration of adsorbed contaminant is always in equilibrium with the concentration of dissolved contaminant. For each contaminant, a single solid/liquid equilibrium distribution constant describes the equilibrium relationship between these concentrations. From Equation 4-1,

$$C_l/N = 1/(k_d + r_{sl}^{-1})$$

where:

C_l = concentration of the contaminant in liquid within the control volume of the impoundment (mg/l or g/m^3)

N = dry weight concentration of contaminant in sludge contained within the control volume (mg/kg)

k_d = equilibrium partition coefficient for the contaminant (l/kg)

r_{sl} = ratio of the mass of solid to the volume of liquid (kg/l)

The partition coefficient k_d describes the ratio between dissolved and adsorbed contaminant concentrations, and is valid at low contaminant concentrations only, where concentrations do not exceed solubility thresholds. For the equations that follow, it is useful to introduce an additional variable k_{sl}, that describes the ratio between adsorbed and dissolved contaminant mass:

$$k_{sl} = k_d \, r_{sl} \qquad (6\text{-}7)$$

where k_{sl} describes the (unitless) ratio of adsorbed contaminant mass to dissolved contaminant mass. Combining Equations 4-1 and 6-7 for $C_l = C_0$ yields:

$$C_0/N = r_{sl}/(1 + k_{sl}) \qquad (6\text{-}8)$$

that describes the concentration of contaminant in the liquid fraction of influent, as a function of the dry-weight concentration of contaminant in inflow. As will be discussed in Sections 6.3.7.1 and 6.3.7.2, it will be assumed that both r_{sl} and k_{sl} are constant within the control volume throughout the active lifetime of a sludge lagoon.

6.3.7. Estimation of Volatile Emissions. Estimation of rates of contaminant volatilization from a surface disposal facility requires estimation of the overall mass transfer coefficient (k) in Equation 6-5, and simultaneous quantification of each of the other contaminant loss processes included in the mass balance described by Equation 6-2. The exact form of the mass balance equation will depend on the type of facility involved. Four types of facilities will be considered: (1) well-mixed impoundments with continuous inflow of sludge or wastewater, (2) poorly mixed impoundments with continuous inflow, (3) sludge impoundments receiving occasional or periodic deposits of sludge and (4) well-mixed impoundments in which sludge accumulates for long-term storage as a product of wastewater treatment. For each type of facility, a mass balance approach yields expected contaminant concentrations in liquid contained within the impoundment. Once these concentrations are determined, total volatilized emissions are easily estimated from Equation 6-4.

6.3.7.1. ESTIMATION OF EMISSIONS FROM WELL-MIXED SLUDGE IMPOUNDMENTS WITH CONTINUOUS INFLOW -- In a well-mixed impoundment, contaminant concentrations are assumed to be constant throughout the entire control volume, and an overall mass balance for the entire facility is used to determine the rate of contaminant emissions. For the liquid phase of contaminant mass,

$$Q\,C_0 + M_{SL} = k\,A\,C_l + k_b\,V\,C_l + QC_l \qquad (6\text{-}9)$$

The first term on the left side of this equation represents the mass of dissolved contaminant entering the control volume per unit of time. The second term represents the mass transferring from adsorbed to liquid phase. On the right side of the equation, the first term represents volatilization from the liquid, the second term represents losses to biodegradation and the final term represents losses through effluent and seepage. For the solids mass balance,

$$k_{sl}\, Q\, C_0 = M_{SL} + k_{sl}\, k_b\, V\, C_l + k_{sl}\, Q\, C_l \tag{6-10}$$

where the term on the left side of the equation gives the mass of adsorbed contaminant flowing into the system. This mass is equal to the contaminant mass in the liquid ($Q\, C_0$) multiplied by a mass partitioning factor (k_{sl}), based on the assumption of local equilibrium between dissolved and adsorbed contaminants within the lagoon. On the right side of the equation, the three terms indicate the mass of contaminant transferring from adsorbed to dissolved phase, the solid contaminant biodegraded and the solid contaminant lost to the system through effluent and settling.

Eliminating M_{sl} from these two equations and solving for the impoundment's concentration yields:

$$C_l = \frac{Q\, C_0\, (1 + k_{sl})}{Q(1 + k_{sl}) + k_b\, V(1 + k_{sl}) + k\, A} \tag{6-11}$$

Substituting into Equation 6-5 gives:

$$E_{site} = \frac{k\, A\, Q\, C_0\, (1 + k_{sl})}{Q(1 + k_{sl}) + k_b\, V(1 + k_{sl}) + k\, A} \tag{6-12}$$

which can also be expressed as:

$$E_{site}/C_0 = \frac{k\,A\,Q\,(1+k_{sl})}{Q(1+k_{sl}) + k_b\,V(1+k_{sl}) + k\,A} \qquad (6\text{-}13)$$

Equation 6-13 gives the estimated rate of volatile emissions (g/sec) as a function of inflow characteristics, and of site- and chemical-specific parameters.

Implicit in Equation 6-10 is the assumption that k_{sl} has the same value for influent to the impoundment, for the impoundment's contents and for effluent and seepage from the impoundment. Since k_{sl} depends on the ratio of suspended solids to liquid (r_{sl}), this assumption further implies that effluent and seepage from the impoundment contain the same fraction of suspended solids as does influent. For most impoundments, effluent will contain lower quantities of suspended solids than influent, since sludge solids will settle and accumulate within sludge lagoons. $QC_l(1+k_{sl})$ is therefore likely to overestimate contaminant losses through effluent and seepage. As mentioned in Section 6.3.5, Equations 6-9 and 6-10 describe activity within a control volume that extends downward from the surface of the impoundment to the top of the sludge layer on the lagoon floor. Conditions within this control volume are assumed to be steady-state, so that liquid concentrations remain constant with time. To the extent that actual contaminant losses from this control volume through effluent, seepage and settling to the lagoon floor can be approximated by $(1+k_{sl})QC_l$, Equation 6-13 may provide a reasonable approximation of volatile emissions from the surface of the lagoon.

6.3.7.2. ESTIMATION OF EMISSIONS FROM POORLY MIXED IMPOUNDMENTS WITH CONTINUOUS INFLOW -- Poorly mixed impoundments are better represented by a "plug flow" model. For these, it is assumed that sludge enters the impoundment at known contaminant concentrations, and flows with time toward the opposite end. Dissipative processes occur along the facility's length and generate location-dependent concentrations. As the mixture flows along the length of the facility, contaminant partitioning and losses via the various pathways described above take place. For this type of impoundment, the mass balance relation is best written for a differential volume element within the lagoon, and the contaminant concentration must be solved as a function of distance from the inlet. The in-liquid contaminant mass balance for a differential volume element in the facility can be described by:

$$Q\,C_l + dM_{SL} = Q(C_l + dC_l) + k\,dA\,C_l + k_b\,dV\,C_l \tag{6-14}$$

where:

$$dA = w\,dx$$

and

$$dV = h\,w\,dx$$

and where:

dA = the surface area of a differential volume element (m^2)
w = the width of the impoundment perpendicular to the direction of flow (m)
dx = the length of a differential volume element
dV = the volume of a differential volume element (m^3)
h = the depth of a volume element (m)

The corresponding mass balance for solid-phase contaminant is described by:

$$k_{sl}\,Q\,C_l = dM_{SL} + k_{sl}\,k_b\,dV\,C_l + [k_{sl}\,Q(C_l + dC_l)] \tag{6-15}$$

where dM_{SL} represents the movement of contaminant mass between solid and liquid phase within the differential volume element (g/sec). Combining these equations and eliminating dM_{SL} yields:

$$(1+k_{sl})Q\,dC_l = -[k\,dA + kb\,dV\,(1+k_{sl})]\,C_l$$

Substituting for the differential elements and rearranging yields:

$$dC_l/dx = \frac{-[k\,w + k_b\,w\,h\,(1+k_{sl})]}{Q(1+k_{sl})}\,C_l$$

This equation can be solved subject to the boundary condition that $C_l(x) = C_0$, to yield concentration as a function of downstream distance (x):

$$C_l(x) = C_0 \exp\left[\frac{-[kw + k_b wh(1+k_{sl})]x}{Q(1+k_{sl})}\right]$$

Substituting into Equation 6-4 yields the rate of volatile emissions as a function of distance:

$$dE_{site}(x) = k\ w\ C_l(x)\ dx$$

To derive total emissions of contaminant for the entire facility, this function is integrated from the inlet (x = 0) to the outlet (x = L), to obtain:

$$E_{site} = k\ A\ C_0\,[1-\exp(-\alpha)]\ /\ \alpha \qquad (6\text{-}16)$$

or

$$E_{site}/C_0 = k\ A\ C_0\,[1-\exp(-\alpha)]\ /\ \alpha \qquad (6\text{-}17)$$

where:

$$\alpha = \frac{k\ A + k_b\ V\ (1+k_{sl})}{Q(1+k_{sl})}$$

As in Section 6.3.7.1, the above equations implicitly assume that effluent and seepage have the same suspended solids content as inflow (i.e., that r_{sl} and k_{sl} are constant throughout the impoundment). To the extent that Q $C_l(L)$ provides a reasonable approximation of total contaminant losses from the control volume through effluent, seepage and settling to the lagoon floor, Equation 6-17 may provide a reasonable estimate of volatilization.

6.3.7.3. **ESTIMATION OF EMISSIONS FROM SLUDGE IMPOUNDMENTS RECEIVING OCCASIONAL OR PERIODIC DEPOSITS OF SLUDGE --** At some facilities, sludge does not enter the impoundment continuously, but rather in discrete deposits. Between deposits of sludge, contaminant mass is degraded or released from the impoundment and concentrations decrease. Contaminant concentrations in the impoundment are therefore a function of elapsed time between sludge deposits, and the mass balance of contaminant within the impoundment is most conveniently described for a discrete time interval between deposits of sludge. If the deposits are sufficiently frequent that emissions from previous deposits are still significant at the time of subsequent deposits, the system is best modeled with methods described in Sections 6.3.7.1 and 6.3.7.2. For those lagoons receiving less frequent deposits of sludge, or those in which sludge is periodically removed, the change of contaminant mass in liquid phase within the volume of sludge deposited can be described as a function of time:

$$- \frac{d(C_l V_{dep})}{dt} + M_{SL} = k\,A\,C_l + k_b\,V_{dep}\,C_l$$

where:

V_{dep} = the volume of sludge deposited (m^3)

t = time elapsed since the sludge was deposited (sec)

For contaminant mass in adsorbed phase,

$$- \frac{d(k_{sl} C_l V_{dep})}{dt} = M_{SL} + k_b\,V_{dep}\,k_{sl}\,C_l$$

If it is assumed that the volume of the liquids in the system remains approximately constant with time, these two relations can be combined to yield

$$\frac{(1+k_{sl})\,V_{dep}\,dC_l}{dt} = -\,[k\,A\,C_l + k_b\,V_{dep}(1+k_{sl})C_l]$$

that can be rearranged to yield an equation for the change in concentration per increment of time:

$$\frac{dC_\ell}{dt} = \frac{-[(k\ A)/V_{dep} + k_b(1+k_{s\ell})]}{1+k_{s\ell}}\ C_\ell$$

This equation can be solved subject to $C_\ell(t=0) = C_0$ to yield

$$C_\ell(t) = C_0 \exp\left[\frac{-[(kA/V_{dep}) + k_b(1+k_{s\ell})]\ t}{(1+k_{s\ell})}\right]$$

or

$$C_\ell(t) = C_0 \exp(-\beta t),$$

where

$$\beta = [(k\ A/V_{dep}) + kb(1+ksl)]\ /\ (1+ksl).$$

It follows by substitution into Equation 6-4 that from time 0 to time T, average emissions can be described by:

$$E_{site} = (1/T) \int_{t=0}^{T} k\ A\ C_\ell(t)\ dt = \frac{C_0 kA\ [1-\exp(-\beta T)]}{\beta T}$$

or

$$E_{site}/C_0 = \frac{kA\ [1-\exp(-\beta T)]}{\beta T} \qquad (6\text{-}18)$$

Several limitations in the above methodology should be noted. First, it was assumed that the volume of liquid contained in the impoundment is constant with time. In many cases, this volume will decrease as the lagoon loses moisture to evaporation, seepage, and effluent. Second, only the most recent deposit of sludge is considered when estimating emissions. In other words, Equation 6-18 estimates the rate at which contaminants are emitted from the most recent deposit of sludge, up until the time when the sludge is removed or the impoundment receives a subsequent deposit. Where sludge deposits are infrequent and rates of volatilization high, this model may provide a reasonable representation of expected emissions. In other applications the model may underestimate actual emissions because of its neglect of potential emissions of contaminants from previous sludge deposits. For impoundments with frequent deposits of sludge, the continuous inflow models described in Section 6.3.7.1 and 6.3.7.2 should be used.

6.3.7.4. ESTIMATION OF EMISSIONS FROM WASTEWATER TREATMENT LAGOONS USED FOR LONG-TERM SLUDGE STORAGE -- Emissions from wastewater treatment lagoons can be modeled with methods similar to those described in Section 6.3.7.1. For both types of facilities, sludge contaminants are assumed to enter the impoundment continuously. Wastewater treatment lagoons differ from sludge impoundments in two respects, however. First, the solids content of liquids entering a wastewater treatment lagoon is typically lower than that of water entering a sludge lagoon. Second, wastewater treatment lagoons may use diffused air as a component of wastewater treatment. The methodology described in Section 6.3.7.1 is equally valid for facilities with lower concentrations of solids in inflow and can be applied to wastewater treatment lagoons that do not use diffused air. This section, therefore, presents a methodology for estimating emissions for wastewater treatment systems that include diffused air.

Diffused air acts as a medium for transporting contaminant directly from the impoundment into the atmosphere. This methodology assumes that contaminant concentrations in diffused air reach equilibrium with the impoundment's liquid before the air reaches the surface. Contaminant emissions are then assumed to equal the volumetric flow rate of diffused air multiplied by the estimated equilibrium concentration in diffused air. As discussed in Section 6.3.4, losses to volatilization in diffused air are assumed to be described by $M_{VD} = Q_a (H_c/RT) C_\ell$. If this term is inserted into Equation 6-8, and Equations 6-9 and 6-10 are combined as in Section 6.3.7.1, the following estimate is obtained for the concentration of contaminant in the impoundment:

$$C_\ell = \frac{Q\, C_0\, (1 + k_{s\ell})}{Q(1+k_{s\ell}) + k_b\, V(1+k_{s\ell}) + kA + Q_a(H_c/RT)} \tag{6-19}$$

Total volatile emissions can then be calculated as:

$$E_{site} = Q_a (H_c/RT)C_l + kAC_l$$

or

$$E_{site}/C_0 = \frac{[Q_a (H_c/RT) + k A]\, Q(1+k_{sl})}{Q(1+k_{sl}) + k_b V(1+k_{sl}) + kA + Q_a(H_c/RT)} \qquad (6\text{-}20)$$

6.3.8. Estimation of Wind Transport

For both Tier 1 and Tier 2 calculations, the methodology includes consideration of the dispersion of contaminants likely to occur during wind transport. This dispersion is estimated with the Industrial Source Complex Long Term (ISCLT) model, as described in U.S. EPA (1979) and U.S. EPA (1986b) and implemented in the Graphical Exposure Modeling System (U.S. EPA, 1988c) and elsewhere. The model predicts long-term average contaminant concentrations in ambient air at specified receptor locations near a site. It assumes that the plume of contaminated air follows a Gaussian distribution, that the emission rate is uniform and continuous over the source, and that meteorological conditions are constant between the source and the receptor location. The model represents the average concentration within 16 directional sectors (each of 22.5°) based on the following:

1. the mass flux from the source area,
2. the wind speed,
3. the distance from the virtual source,
4. the vertical standard deviation of the Gaussian plume based on the stability class and
5. deposition flux onto the ground surface and losses to other decay processes.

For estimating contaminant transport from area sources of emissions (such as surface impoundments), the model represents emissions as a point source located at a specified virtual distance upwind of the actual site. From ESE (1985), the atmospheric transport and dispersion model for the sector-averaged concentration of a specific sector near an area source of emissions is:

$$C_a(X_a,Z_a) = \sum 2.03 \frac{E_{site}A}{\sigma_z(X_a+x_{vi})} \exp \frac{-Z_a^{2}}{2\sigma_z^{2}} \sum \frac{f_{ij}}{U_{wj}} \qquad (6\text{-}21)$$

where:

C_a = the concentration of contaminant in air at the location defined by X_a and Z_a

X_a = distance in x-coordinate direction (parallel to velocity U_{wj}) from source to point of interest (m)

Z_a = distance in z-coordinate direction (perpendicular to wind velocity, U_{wj}) from source to point of interest (m)

E_{site} = mass flux of contaminant into the atmosphere (gm/sec)

Λ = first-order source depletion due to atmospheric decay of contaminant, and wet and dry deposition (dimensionless)

σ_z = mixing coefficient in z direction, standard deviation of Gaussian plume (m)

x_{vi} = virtual distance required for point source plume to spread to width of site (m)

f_{ij} = frequency of the specific stability array parameters for classification i,j (stability class, wind speed)

U_{wj} = average wind speed in j direction (m/sec)

First-order contaminant losses are calculated as $\Lambda = \exp[-\lambda(X_a/U_w)]$, where λ equals a lumped first-order loss constant (sec^{-1}) and X_a/U_w equals the time required for transport of the contaminant from the site to the receptor location. According to ESE (1985), contaminant losses to deposition and atmospheric decay are usually negligible within the time required to reach the location of the most exposed individual. The mixing coefficient in the vertical direction (σ_z) is calculated from the distance X_a, based on parameters that vary according to Pasquill-Guifford stability category. Parameter values appropriate for stable atmospheric conditions, for example, are presented in Table 6-2.

The input requirements for ISCLT can be satisfied with local stability array (STAR) weather data available for most locations in the United States. For each radial distance specified by the user, the model provides estimated average ground level concentrations of contaminant in each of 16 directions from the source. For an analysis of exposure to an MEI, the modeler is interested in the highest predicted air concentration beyond the property boundary. Because Equation 6-21 is linear with respect to E_{site}, the ratio between air concentration and emissions will be constant for a single contaminant, a specified lagoon location and area, and a single receptor location, so that a "wind transport ratio" can be defined as C_a/E_{site} for the MEI.

6.3.9. Deriving Criteria. The final step in deriving Tier 2 criteria for the air pathway is to relate the selected RAC at the location of the MEI to the maximum allowable dry-weight

TABLE 6-2

Parameters Used to Calculate σ_z[a]

Pasquil Stability Category	x (km)	$\sigma_z = \alpha x^\beta$	
		α	β
Very Unstable[b]	0.10 - 0.15	158.080	1.04520
	0.16 - 0.20	170.222	1.09320
	0.21 - 0.25	179.520	1.12620
	0.26 - 0.30	217.410	1.26440
	0.31 - 0.40	258.890	1.40940
	0.41 - 0.50	346.750	1.72830
	0.51 - 3.11	453.850	2.11660
	3.11	(c)	(c)
Unstable[b]	0.10 - 0.20	90.673	0.93198
	0.21 - 0.40	98.483	0.98332
	0.40	109.300	1.09710
Slightly Unstable[b]	0.10	62.141	0.91465
Neutral	0.10 - 0.30	34.458	0.86974
	0.31 - 1.00	32.093	0.81066
	1.01 - 3.00	32.093	0.64403
	3.01 - 10.0	33.504	0.60586
	10.01 - 30.0	36.650	0.56589
	30.00	44.053	0.51179
Slightly Stable	0.10 - 0.30	23.331	0.81956
	0.31 - 1.00	21.628	0.75660
	1.01 - 2.00	21.628	0.63077
	2.01 - 4.00	22.534	0.57154
	4.01 - 10.0	24.703	0.50527
	10.01 - 20.0	26.970	0.46713
	20.01 - 40.0	35.420	0.37615
	40.00	47.618	0.29592

TABLE 6-2 (cont.)

Pasquil Stability Category	x (km)	$\sigma_z = \alpha x^\beta$	
		α	β
Stable	0.10 - 0.20	15.209	0.81558
	0.21 - 0.70	14.457	0.78407
	0.71 - 1.00	13.953	0.68465
	1.01 - 2.00	13.953	0.63227
	2.01 - 3.00	14.823	0.54503
	3.01 - 7.00	16.187	0.46490
	7.01 - 15.0	17.836	0.41507
	15.01 - 30.0	22.651	0.32681
	30.01 - 60.0	27.084	0.27436
	> 60.00	34.219	0.21716

[a] Source: Environmental Science and Engineering, 1985
[b] If the calculated value of σ_z exceeds 5000 m, σ_z is set to 5000 m
[c] σ_z=5000 m

concentrations of contaminant in sludge received by the impoundment. As for other pathways discussed in this document, this relation may be described by a "source receptor ratio" (in kg/m^3) for each impoundment site and MEI location evaluated:

$$SRR_{VOL} = C_a/N \tag{6-25}$$

where C_a represents the expected concentration of contaminant in ambient air (mg/m^3) at the location of the MEI, and N represents the concentration of a contaminant in sludge (mg/kg). For each sludge contaminant to be considered, this ratio is easily calculated with Equations 6-22 and 6-8, together with an estimate of E_{site}/C_0 derived by Equation 6-13, 6-17, 6-18 or 6-20:

$$SRR_{VOL} = (C_a/E_{site})(E_{site}/C_0)(C_0/N) \tag{6-26}$$

Once SRR_{VOL} has been estimated from Equation 6-26, maximum allowable levels of each contaminant in sludge received or accumulated in a surface disposal facility (in mg/kg dry-weight) can be determined by:

$$N_{max} = (RAC\ mg/m^3)/(SRR_{VOL}\ kg/m^3) \tag{6-27}$$

A worst-case scenario for wind transport of emitted contaminants, appropriate for Tier 1 criteria, could be created with results from an ISCLT run based on STAR data for Santa Barbara, California, which produces the highest concentrations at the receptor site. With the conservative assumption that contaminant loss through degradation is negligible during wind transport to the location of the MEI, the transport ratio is the same for all volatile contaminants for a given site and a given receptor location. For this reason, a single estimated value for the transport ratio (or perhaps different values for different ranges of wind speed) could be used to derive all Tier 1 criteria.

A slightly different approach for deriving a transport ratio for Tier 1 would be based on a set of simplifying assumptions described in ESE (1985) and incorporated into the methodology described in U.S. EPA (1986a). This approach assumes (1) that there are no contaminant losses to deposition or other atmospheric decay processes ($\Lambda = 1$), (2) that the wind direction is constant in the direction of the receptor, (3) that atmospheric conditions are always stable with no restriction on vertical mixing, (4) that the receptor is located on the centerline of wind direction ($Y_a = 0$), (5) that the receptor breathes air at concentrations predicted for ground level ($Z_a = 0$) and (6) that wind speed is constant, described by the local average ($U_j = U_w$ for all j). With these assumptions, Equation 6-21 reduces to:

$$C_a = 2.032\ E_{site} / [\sigma_z\ (X_a + x_{vi})\ U_w] \qquad (6\text{-}23)$$

All of the above assumptions lead to a conservative estimate of average downwind contaminant concentrations. The assumption of unlimited vertical mixing could lead to a lower estimate of contaminant concentrations at the receptor location, but should not affect results under stable conditions and over the short transport distances to be considered when deriving criteria. For a more precise calculation, the full ISCLT procedure should be executed and site-specific stability array (STAR) weather data should be used.

ESE (1985) recommends approximating x_{vi} by assuming that all contaminant is contained in a 22.5° angular sector that originates at the virtual source and for which the width matches the effective diameter of the site. Based on this approach, virtual distance can be approximated by:

$$x_{vi} = \frac{d_e}{2} \cot(22.5/2)$$

where $d_e = 2(A/\pi)^{0.5}$ equals the effective diameter of the lagoon. This equation can be approximated by $x_{vi} = 2.84\ A^{0.5}$, and produces a conservative derivation of virtual distance when compared with a more precise method based on atmospheric conditions and wind velocity. A nearly identical approach has been found to give reasonable predictions of ambient air concentrations under experimental conditions (Baker and MacKay, 1985).

Equation 6-23 can be rearranged as:

$$C_a/E_{site} = 2.032 \ / \ [\sigma_z \ (X_a+x_{vi})U_w] \qquad (6\text{-}24)$$

6.4. INPUT PARAMETER REQUIREMENTS

Several input parameters are needed to describe conditions within the impoundment to be modeled, and to describe meteorological conditions in the surrounding area. Reasonable worst case parameter values should be used for deriving Tier 1 numerical criteria for national application, but values appropriate for each individual site must be specified for Tier 2. Three types of parameters are required: (1) those describing the chemical or physical characteristics of the volatile contaminant under consideration, (2) those describing the surface disposal facility and the quantity of wastewater or sludge it receives, and (3) meteorological characteristics of the area.

6.4.1. Chemical Characteristics. For each sludge contaminant to be modeled, the user must specify four parameters. The diffusivity of the contaminant in water and its diffusivity in air may be derived with methods described in Lyman (1982). Henry's Law constant can be obtained from Table 5-2. Biodegradation rates for many organic chemicals are tabulated in Schnoor et al. (1987). Partition coefficients for each contaminant can be derived with Equation 5-15, using values of K_{ow} from Table 5-2, or can be derived from K_{oc} values reported in Appendix C of U.S. EPA (1986a), or can be estimated with the CHEMEST procedures in GEMS (U.S. EPA, 1988c).

6.4.2. Site Characteristics. Dimensions of each lagoon, rates of influent and effluent flows, and rates of diffused air flow should be readily available at each site. The distance to the property boundary can be determined from the site plan. Rates of seepage from the facility can be estimated from a mass balance that includes inflow, effluent, evaporation and precipitation. The concentration of solids in sludge or wastewater received by each facility can be obtained from site measurements.

6.4.3. Meteorological Conditions. Local average temperature, wind speeds, and atmospheric stability data can be obtained from the STAR station nearest to each site.

6.5. HEALTH AND ENVIRONMENTAL EFFECTS

Toxic pollutants that volatilize to air from surface disposal sites can cause adverse human health effects to those living downwind from the surface disposal site. The reference air concentration (RAC) is defined as the air concentration of pollutant used to evaluate the potential for adverse effects on human health. If a particular concentration of pollutant in sludge results in an air concentration that is greater than the RAC, adverse health effects may occur in a population exposed to this concentration at a compliance point downwind from the disposal site.

The units used for the RAC in this methodology are mg/m^3. To obtain the RAC in these units, a reference intake in mg-day/kg is converted using a particular human body weight assumed to represent the average in the exposed population, and using an assumed average daily air inhalation rate. Furthermore, the reference intake is adjusted to account for the intake of the contaminant from sources unrelated to contamination from sludge.

The procedure for determining the RAC varies according to whether the pollutant acts by a threshold or nonthreshold mechanism of toxicity.

6.5.1. Threshold-Acting Toxicants. Threshold-acting toxicants are those for which a dose can be identified below which no adverse effects are assumed to occur. The Agency assumes that all noncarcinogenic chemicals act according to threshold mechanisms. The RAC is derived as follows for threshold-acting toxicants:

$$\text{RAC} = [(\text{RfD BW RE}^{-1}) - \text{TBI}] / I_a \qquad (6\text{-}28)$$

where:

RAC = reference air concentration (mg/m^3)
RfD = reference dose (mg-day/kg)
BW = human body weight (kg)
RE = relative effectiveness of inhalation exposure (unitless)
TBI = total background intake rate of contaminant (mg/day) from all other sources of exposure
I_a = total air inhalation rate (m^3/day)

The parameters RfD, BW, and TBI have been discussed in previous sections. The definition and derivation of I_a and RE are given below.

6.5.1.1. TOTAL AIR INHALATION RATE (I_a) -- Table 6-3 shows values of I_a for a typical man, woman, child and infant with a typical activity schedule, as defined by the International Commission on Radiological Protection (ICRP, 1975). The Agency has used an assumption of 20 m^3 per day for a 70-kg adult to represent a typical inhalation rate in many Agency risk assessments. Values have also been derived for two additional scenarios: an adult with the same activity schedule, but with upper limit rather than average respiration rates for each activity, and an adult with normal respiration rates but whose work is moderately active and who practices 1 hour of heavy activity per day (Fruhman, 1964; Astrand and Rodahl, 1977). Representative body weights have been assigned to each of these individuals to derive a respiratory volume-to-body weight ratio. The resulting ratios range from 0.33 to 0.47 m^3-day/kg. Values in this range exceed the ratio value of 0.29 m^3-day/kg, which corresponds to a 70-kg adult inhaling 20 m^3 per day. Therefore, use of the latter value may underestimate actual exposure for certain adults. In circumstances where children are known to be at special risk, it may be more appropriate to base the derivation of criteria on the inhalation rate of toddlers or infants.

6.5.1.2. FRACTION OF INHALED AIR FROM CONTAMINATED AREA -- All individuals exposed to emissions from a surface disposal site may not necessarily remain in proximity to the surface disposal site for 24 hours a day. However, if residential areas near the site are affected, then less mobile individuals (e.g., infants or the elderly) may be exposed for approximately 24 hours a day. These less mobile residents may be included among those determined to be at greatest risk, whom the criteria are designed to protect. Therefore, it is reasonable to assume for this methodology that 100% of the air inhaled by the MEI is from the area of the surface disposal site.

6.5.1.3. RELATIVE EFFECTIVENESS OF EXPOSURE (RE) -- RE, as discussed previously, is a unitless factor that shows that relative toxicological effectiveness of an exposure by a given route when compared with another route. Since exposure from the vapor pathway is through inhalation, the RE factors applied in Equation 6-25 represent the relative effectiveness of exposure though the media for which the RfD was derived when compared with exposure through inhalation. Where no relevant data are available, RE should be assumed to equal 1.

TABLE 6-3

Daily Respiratory Volumes for "Reference" Individuals (Normal Individuals at Typical Activity Levels) and for Adults with Higher-than-Normal Respiratory Volume or Higher-than-Normal Activity Levels[a]

	"Reference" Man[b] (m^3/day)	"Reference" Woman[b] (m^3/day)	Upper-Limit Adult (m^3/day)	Active Adult (m^3/day)	"Reference" Child: 10 years[b] (m^3/day)	"Reference" Infant: 1 year[b] (m^3/day)
8-hour working, light activity	9.6	9.1	12.0[c]	--	6.24	2.5 (10 hours)
8-hour working, moderate activity	--	--	--	14.4[d]	--	--
8-hour nonoccupational (light) activity	9.6	9.1	12.0[c]	--	6.24	--
7-hour nonoccupational (light) activity	--	--	--	8.4	--	--
1-hour heavy activity	--	--	--	2.5[d]	--	--
8-hour rest	3.6	2.9	7.0[e]	3.6	2.3	1.3 (14 hours)
Total daily respiratory volume	22.8	21.1	31.0	28.9	14.78	3.8
Body weight (kg)	70	58	65	65	33	10
Ratio of volume to body weight[f]	0.33	0.36	0.47	0.44	0.45	0.38

[a] Source: U.S. EPA, 1989d.
[b] Upper-limit values for "ordinary man or woman," for "light" activity category (Astrand and Rodahl, 1977).
[c] Averaged values for "ordinary man or woman," for "moderate" or "heavy" activity categories (Astrand and Rodahl, 1977)
[d] Upper 95th percentile for adults, age 20-39 years, at rest (Fruhman, 1964)
[e] The inhalation volume to body weight ratios have been derived from referenced values for illustrative purposes only.

6.5.2. Carcinogens. The general approach used by the Agency to assess carcinogens is discussed in Section 4.5.2. For pollutants assessed as carcinogenic (nonthreshold-acting) agents, the reference air concentration is derived as follows:

$$RAC = (RL\ BW) / (q_1^*\ RE\ I_a) \tag{6-29}$$

where:

RAC = reference air concentration (mg/m^3)
RL = risk level (unitless)
BW = human body weight (kg)
q_1^* = human cancer potency [(mg-day/kg)$^{-1}$]
RE = relative effectiveness of inhalation exposure (unitless)
I_a = total air inhalation rate (m^3/day)

For carcinogens, the RAC as computed with Equation 6-29 is thought to be protective, since the estimate of carcinogenicity is an upper limit value. All parameters have been defined and discussed in previous sections.

6.6. SAMPLE CALCULATIONS

Sample calculations for benzene volatilization are provided for three actual surface disposal facilities: a wastewater treatment and sludge storage facility in Antrim, New Hampshire, a sludge storage facility in Tulsa, Oklahoma, and a sludge disposal facility in Portland, Oregon.

6.6.1. Analysis of Exposure for the Most Exposed Individual. Sample calculations will be presented in this section for the derivation of criteria for the same three sites evaluated in Sections 4.7, and 5.6. These sample calculations determine the maximum allowable concentration of benzene for sludge that accumulates or is deposited in these surface disposal facilities. Actual or estimated benzene concentrations in the sludge are then compared with this value to determine whether sludge characteristics are acceptable. Benzene has been chosen to illustrate the methodology because it is highly volatile and because exposure via the air pathway may be of concern. Pertinent chemical-specific data for benzene are provided in Table 6-4. It is assumed that the air pathway is not of concern for lead, the other contaminant evaluated in Sections 4.7 and 5.5.

TABLE 6-4

Input Parameters for Estimating Emissions of Benzene

Parameter	Value	(Units)
μ_a	1.8×10^{-4}	(g/cm-sec)
ρ_a	1.2×10^{-3}	(g/cm^3)
D_{ca}	8.8×10^{-2}	(cm^2/sec)
D_{cw}	9.8×10^{-6}	(cm^2/sec)
D_{eth}	8.5×10^{-6}	(cm^2/sec)
H_c	5.5×10^{-3}	(atm-m^3/mol)
R	8.21×10^{-5}	(m^3-atm/mol-°K)
Sc_G	1.7	(unitless)
Sc_L	918	(unitless)
f_{oc}	0.50	(unitless)
k_{oc}	74	(*l*/kg)
k_d	37	(*l*/kg)
$[k_b]_{20}$	1.27×10^{-6}	(sec^{-1})
$[k_b]_{25}$	1.7×10^{-6}	(sec^{-1})

6.6.1.1. SAMPLE CALCULATION: ANTRIM, NEW HAMPSHIRE -- The three lagoons at the Antrim wastewater treatment facility were described in Section 4.7.1.1. For the calculations carried out in this section, it is assumed that the contents of the lagoons are well-mixed: that the characteristics of the sludge and liquid are similar throughout the lagoons. The three lagoons are modeled as a single aggregated facility with total area equal to the total of the three impoundments. The calculations proceed in six steps:

1. Estimation of the reference air concentration (RAC, in mg/m^3) for benzene,
2. Estimation of the ratio between the concentration of dissolved contaminant in the lagoon and the dry-weight concentration in sludge received by or accumulated in the lagoon (C_0/N, in kg/m^3)
3. Estimation of the ratio of emissions to contaminant concentrations in liquid contained in the lagoon (E_{site}/C_l, in m^3/sec)
4. Estimation of the ratio between concentrations of contaminant in ambient air at the selected receptor location for the MEI and the rate of total emissions from the site (C_a/E_{site}, in sec/m^3),
5. Estimation of the ratio between contaminant concentrations in ambient air and dry weight concentrations in sludge (SRR_{VOL}, in kg/m^3), and
6. Estimation of the maximum allowable dry-weight concentration of contaminant in sludge received by or accumulated in the surface disposal facility (N_{max}, in mg/kg).

Each of these steps will be described below.

6.6.1.1.1. Derivation of the Reference Air Concentration -- Tier 1 and Tier 2 calculations begin with the derivation of a reference air concentration, or RAC. Since benzene is a carcinogen with an established potency value (q_l^*), the reference air concentration is calculated with Equation 6-29:

$$RAC = (RL\ BW) / (q_1^*\ RE\ I_a)$$

For illustration, this sample calculation arbitrarily selects a risk level of 10^{-6}. Based on a body weight of 70 kg, an inhalation volume of 20 m^3/day and a relative effectiveness of 1.0 for inhalation,

$$RAC = [(10^{-6})(70)] / [(2.9x10^{-2})(1)(20)] = 1.2 \times 10^{-4} \text{ mg/m}^3$$

6.6.1.1.2. **Estimation of C_0/N** -- The relationship between the concentration of the benzene in the inflowing liquid and the dry-weight concentration of benzene in sludge accumulating in the impoundment is estimated with Equation 4-1 based on a reported solids concentration of $5.4x10^{-4}$ kg/l in influent to the impoundment:

$$C_0/N = 1/(k_d + r_{sl}^{-1}) = 1/[37+(5.3x10^{-4})^{-1}] = 5.3x10^{-4} \text{ kg}/l = 0.53 \text{ kg/m}^3$$

6.6.1.1.3. **Estimation of E_{site}/C_0** -- Expected emissions of benzene from the impoundment are next related to the concentration of benzene in the lagoons' inflow, based on Equation 6-19 for well-mixed systems with diffused air. Evaluation of that equation requires estimation of the overall mass transfer coefficient. This coefficient is estimated from the liquid and gas phase mass transfer coefficients, from Equation 6-5:

$$\frac{1}{k} = \frac{1}{k_\ell} + \frac{RT}{H_c k_g}$$

Selection of an equation for estimating k_ℓ requires calculation of the effective diameter (d_e) or "fetch" of the idealized lagoon:

$$d_e = 2(A/\pi)^{0.5} = 2(5040/3.142)^{0.5} = 80.1 \text{ m}$$

Since the depth of the lagoons is approximately 3 meters, the fetch:depth ratio is about (80/3)=26. Since average wind speed in Antrim is 3.2 m/sec (less than 3.25 m/sec), and the fetch:depth ratio for the Antrim facility is 26 (which is between 14 and 52), k_ℓ can be calculated from Equation 6-21:

$$k_\ell = 2.78\text{x}10^{-6}\,(D_w/D_{eth})^{2/3}$$
$$= 2.78\text{x}10^{-6}\,(9.8\text{x}10^{-6}/8.5\text{x}10^{-6})^{2/3} = 3.1\text{x}10^{-6}\ \text{m/sec}$$

The mass transfer coefficient for the gas side requires estimation of the Schmidt number for the contaminant in air:

$$Sc_G = \mu_a/(\rho_a\,D_{ca})$$
$$= 1.8\text{x}10^{-4}/[(1.2\text{x}10^{-3})(8.8\text{x}10^{-2})] = 1.7$$

Substituting into Equation 5-24 yields:

$$k_g = 1.8\text{x}10^{-3}\,U_w^{0.78}\,Sc_g^{-0.67}\,d_e^{-0.11}$$
$$= (1.8\text{x}10^{-3})(3.2)^{0.78}(1.7)^{-0.67}(80)^{-0.11}$$
$$= 1.9\text{x}10^{-3}\ \text{m/sec}$$

Substituting values for k_ℓ and k_g into Equation 6-6 yields

$$1/k = 1/(3.1\text{x}10^{-6}) + (8.21\text{x}10^{-5})(298)/[(5.5\text{x}10^{-3})(1.9\text{x}10^{-3})]$$

or

$$k = 3.0\text{x}10^{-6}\ \text{m/sec}$$

The biodegradation rate for Equation 6-20 is adjusted to a temperature of 25° C with Equation 5-27:

$$k_b = 1.27\text{x}10^{-6}\,(1.06)^{(25-20)} = 1.7\text{x}10^{-6}\ \text{sec}^{-1}$$

From Equation 6-20 and input values from Tables 6-4 and 6-5, emissions can be related to contaminant concentrations in inflow to the lagoon:

$$
\begin{aligned}
E_{site}/C_0 &= \frac{[Q_a\,(H_c/RT) + k\,A]\,Q(1+k_{sl})}{\{Q(1+k_{sl}) + k_bV(1+k_{sl}) + kA + Q_a(H_c/RT)\}} \\
&= \frac{[(6.2)(0.22) + (3.0\text{x}10^{-6})(5.040)](0.01)(1.02)}{(0.01)(1.02) + (1.7\text{x}10^{-6})(1.5\text{x}10^{4})(1.02)+(3.0\text{x}10^{-6})(5040)+(6.14)(0.22)} \\
&= 9.9\text{x}10^{-3}\ \text{m}^3/\text{sec}
\end{aligned}
$$

6.6.1.1.4. Estimation of the Transport Ratio (C_a/E_{site}) -- To estimate the expected dispersion of benzene during wind transport, the ISCLT model was executed with STAR data retrieved through GEMS from the Concord, NH airport (latitude 43.2, longitude 71.5). These data are converted into a form usable by ISCLT, as presented in Table 6-6. Execution of ISCLT with these input values results a maximum concentration of benzene in ambient air beyond 500 meters of the site of $4.6\text{x}10^{-2}$ g/m^3 for each g/m^2-sec emitted from the site. This unit rate of emissions is equivalent to 5040 g/sec from a site of 5040 m^2, so:

$$C_a/E_{site} = (4.6\text{x}10^{-2}\ \text{g/m}^2)/(5040\ \text{g/sec}) = 9.2\text{x}10^{-6}\ \text{sec/m}^3$$

6.6.1.1.5. Estimation of the Source-Receptor Ratio (SRR_{VOL}) --
Results derived in Sections 6.6.1.1.2 through 6.6.1.1.4 (and summarized in Table 6-5) are combined to estimate the source-receptor ratio, or C_a/N:

$$
\begin{aligned}
SRR_{VOL} &= (C_a/E_{site})(E_{site}/C_0)(C_0/N) \\
&= (9.2\text{x}10^{-6}\ \text{sec/m}^3)(9.9\text{x}10^{-3}\ \text{m}^3/\text{sec})(0.53\ \text{kg/m}^3) \\
&= 4.8\text{x}10^{-8}\ \text{kg/m}^3
\end{aligned}
$$

6.6.1.1.6. Estimation of the Maximum Allowable Concentration in Sludge -- From Equation 6-27, the maximum allowable dry-weight concentration of benzene in sludge accumulating in the Antrim facility is then:

TABLE 6-5

Derivation of Criteria for the Air Pathway
Antrim, New Hampshire

Parameter	Value	(Units)
A	5040	(m^2)
h	3.0	(m)
V	15000	(m^3)
Q	0.01	(m^3/sec)
Q_a	6.14	(m^3/sec)
r_{sl}	0.54	(kg/m^3)
k_{sl}	0.02	(unitless)
U_{10}	3.2	(m/sec)
T	298	(°K)
d_e	80	(m)
U^*	0.091	(m/sec)
k_l	3.1×10^{-6}	(m/sec)
k_g	1.9×10^{-3}	(m/sec)
k	3.0×10^{-6}	(m/sec)
C_0/N	0.53	(kg/m^3)
E_{site}/C_0	9.9×10^{-3}	(m^3/sec)
C_a/E_{site}	9.2×10^{-6}	(sec/m^3)
SRR_{vol}	4.8×10^{-8}	(kg/m^3)
N_{max}	2500	(mg/kg)

TABLE 6-6

Input Parameters for Execution of ISCLT

Number of Source	1
Number of X Axis Grid System Points	30
Number of Y Axis Grid System Points	16
Number of Special Points	0
Number of Seasons	1
Number of Wind Speed Classes	6
Number of Stability Classes	6
Number of Wind Direction Classes	16
File Number of Data File Used for Reports	1
Program Mode	Rural
Concentration (Deposition) Units Conversion Factor	1.0×10^{6}
Acceleration of Gravity (meters/sec^2)	9.800
Height of Measurement of Wind Speed (meters)	10.000
Correction Angle for Grid System Versus Direction Data North (degrees)	0.000
Decay Coefficient	0.1155×10^{-2}
Program option switches:	1, 2, 2, 0, 0, 3, 2, 3, 3, 0, 0, 0, 7,-8,-9, 0, 0, 1, 0, 0, 1, 1, 1, 1
Width of Area (M)	71.0
Source Strength (g/sec-m^2)	1.0

TABLE 6-6 (cont.)

Ambient Air Temperature (Degrees Kelvin)

	Stability Category 1	Stability Category 2	Stability Category 3	Stability Category 4	Stability Category 5	Stability Category 6
Season 1	283.9000	283.9000	283.9000	280.6000	277.6000	277.6000

TABLE 6-6 (cont.)

Mixing Layer Height (meters)

	Wind Speed Category 1	Wind Speed Category 2	Wind Speed Category 3	Wind Speed Category 4	Wind Speed Category 5	Wind Speed Category 6
Stab. Category 1	0.18225×10^4	0.18225×10^4	0.18225×10^4	0.18225×10^4	0.18225×10^4	0.18225×10^4
Stab. Category 2	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4
Stab. Category 3	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4
Stab. Category 4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4	0.12150×10^4
Stab. Category 5	0.10000×10^5	0.10000×10^5	0.10000×10^5	0.10000×10^5	0.10000×10^5	0.10000×10^5
Stab. Category 6	0.10000×10^5	0.10000×10^5	0.10000×10^5	0.10000×10^5	0.10000×10^5	0.10000×10^5

TABLE 6-6 (cont.)

Frequency of Occurrence of Wind Speed, Direction and Stability
Stability Category 1

Direction (degrees)	Wind Speed Category 1 (1.5 mps)	2 (2.5 mps)	3 (4.3 mps)	4 (6.8 mps)	5 (9.5 mps)	6 (12.5 mps)
0.000	0.00022001	0.00016000	0.00000000	0.00000000	0.00000000	0.00000000
22.500	0.00002000	0.00002000	0.00000000	0.00000000	0.00000000	0.00000000
45.000	0.00012000	0.00009000	0.00000000	0.00000000	0.00000000	0.00000000
67.500	0.00008000	0.00005000	0.00000000	0.00000000	0.00000000	0.00000000
90.000	0.00013000	0.00016000	0.00000000	0.00000000	0.00000000	0.00000000
112.500	0.00033001	0.00030001	0.00000000	0.00000000	0.00000000	0.00000000
135.000	0.00030001	0.00021001	0.00000000	0.00000000	0.00000000	0.00000000
157.500	0.00018001	0.00011000	0.00000000	0.00000000	0.00000000	0.00000000
180.000	0.00058002	0.00030001	0.00000000	0.00000000	0.00000000	0.00000000
202.500	0.00023001	0.00023001	0.00000000	0.00000000	0.00000000	0.00000000
225.000	0.00052002	0.00027001	0.00000000	0.00000000	0.00000000	0.00000000
247.500	0.00032001	0.00018001	0.00000000	0.00000000	0.00000000	0.00000000
270.000	0.00051002	0.00037001	0.00000000	0.00000000	0.00000000	0.00000000
292.500	0.00033001	0.00025001	0.00000000	0.00000000	0.00000000	0.00000000
315.000	0.00042001	0.00041001	0.00000000	0.00000000	0.00000000	0.00000000
337.500	0.00023001	0.00018001	0.00000000	0.00000000	0.00000000	0.00000000

TABLE 6-6 (cont.)

Frequency of Occurrence of Wind Speed, Direction and Stability
Stability Category 2

Direction (degrees)	Wind Speed Category 1 (1.5 mps)	2 (2.5 mps)	3 (4.3 mps)	4 (6.8 mps)	5 (9.5 mps)	6 (12.5 mps)
0.000	0.00209006	0.00128004	0.00043001	0.00000000	0.00000000	0.00000000
22.500	0.00105003	0.00050002	0.00009000	0.00000000	0.00000000	0.00000000
45.000	0.00108003	0.00048001	0.00027001	0.00000000	0.00000000	0.00000000
67.500	0.00097003	0.00039001	0.00027001	0.00000000	0.00000000	0.00000000
90.000	0.00123004	0.00055002	0.00027001	0.00000000	0.00000000	0.00000000
112.500	0.00127004	0.00048001	0.00021001	0.00000000	0.00000000	0.00000000
135.000	0.00173005	0.00112003	0.00059002	0.00000000	0.00000000	0.00000000
157.500	0.00149004	0.00123004	0.00080002	0.00000000	0.00000000	0.00000000
180.000	0.00214006	0.00201006	0.00126004	0.00000000	0.00000000	0.00000000
202.500	0.00197006	0.00146004	0.00075002	0.00000000	0.00000000	0.00000000
225.000	0.00313009	0.00237007	0.00121004	0.00000000	0.00000000	0.00000000
247.500	0.00223007	0.00176005	0.00103003	0.00000000	0.00000000	0.00000000
270.000	0.00284009	0.00237007	0.00128004	0.00000000	0.00000000	0.00000000
292.500	0.00279008	0.00210006	0.00185006	0.00000000	0.00000000	0.00000000
315.000	0.00379011	0.00263008	0.00183005	0.00000000	0.00000000	0.00000000
337.500	0.00309009	0.00196006	0.00100003	0.00000000	0.00000000	0.00000000

TABLE 6-6 (cont.)

Frequency of Occurrence of Wind Speed, Direction and Stability
Stability Category 3

Direction (degrees)	Wind Speed Category					
	1 (1.5 mps)	2 (2.5 mps)	3 (4.3 mps)	4 (6.8 mps)	5 (9.5 mps)	6 (12.5 mps)
0.000	0.00126004	0.00146004	0.00151005	0.00027001	0.00000000	0.00000000
22.500	0.00062002	0.00064002	0.00064002	0.00005000	0.00000000	0.00000000
45.000	0.00090003	0.00075002	0.00094003	0.00000000	0.00000000	0.00000000
67.500	0.00061002	0.00043001	0.00059002	0.00009000	0.00000000	0.00000000
90.000	0.00076002	0.00080002	0.00071002	0.00000000	0.00002000	0.00000000
112.500	0.00059002	0.00048001	0.00089003	0.00025001	0.00000000	0.00000000
135.000	0.00115003	0.00148004	0.00247007	0.00016000	0.00002000	0.00000000
157.500	0.00144004	0.00153005	0.00304009	0.00027001	0.00000000	0.00000000
180.000	0.00157005	0.00201006	0.00288009	0.00043001	0.00000000	0.00000000
202.500	0.00104003	0.00162005	0.00222007	0.00023001	0.00000000	0.00000000
225.000	0.00156005	0.00267008	0.00354011	0.00057002	0.00000000	0.00000000
247.500	0.00101003	0.00153005	0.00354011	0.00050002	0.00002000	0.00000000
270.000	0.00133004	0.00201006	0.00452014	0.00059002	0.00002000	0.00000000
292.500	0.00195006	0.00301009	0.00621019	0.00119004	0.00009000	0.00005000
315.000	0.00305009	0.00562017	0.00916027	0.00212006	0.00023001	0.00002000
337.500	0.00211006	0.00313009	0.00438013	0.00114003	0.00009000	0.00002000

TABLE 6-6 (cont.)

Frequency of Occurrence of Wind Speed, Direction and Stability
Stability Category 4

Direction (degrees)	Wind Speed Category 1 (1.5 mps)	2 (2.5 mps)	3 (4.3 mps)	4 (6.8 mps)	5 (9.5 mps)	6 (12.5 mps)
0.000	0.00701021	0.00797024	0.00573017	0.00375011	0.00030001	0.00000000
22.500	0.00470014	0.00591018	0.00537016	0.00206006	0.00014000	0.00005000
45.000	0.00778023	0.00920028	0.00820025	0.00283008	0.00034001	0.00007000
67.500	0.00418013	0.00523016	0.00525016	0.00244007	0.00037001	0.00032001
90.000	0.00560017	0.00649020	0.00537016	0.00267008	0.00030001	0.00005000
112.500	0.00462014	0.00553017	0.00502015	0.00233007	0.00018001	0.00002000
135.000	0.00857026	0.00993030	0.00767023	0.00272008	0.00016000	0.00000000
157.500	0.00625019	0.00895027	0.01062032	0.00308009	0.00009000	0.00000000
180.000	0.00654020	0.00929028	0.01023031	0.00343010	0.00014000	0.00002000
202.500	0.00266008	0.00361011	0.00514015	0.00233007	0.00011000	0.00002000
225.000	0.00337010	0.00413012	0.00468014	0.00276008	0.00009000	0.00002000
247.500	0.00222007	0.00331010	0.00498015	0.00411012	0.00014000	0.00009000
270.000	0.00269008	0.00377011	0.00728022	0.00808024	0.00112003	0.00005000
292.500	0.00440013	0.00596018	0.01238037	0.02206066	0.00393012	0.00057002
315.000	0.00992030	0.01304039	0.01984060	0.03494105	0.00820025	0.00210006
337.500	0.00629019	0.00824025	0.01028031	0.01886057	0.00377011	0.00057002

TABLE 6-6 (cont.)

Frequency of Occurrence of Wind Speed, Direction and Stability
Stability Category 5

Direction (degrees)	Wind Speed Category					
	1 (1.5 mps)	2 (2.5 mps)	3 (4.3 mps)	4 (6.8 mps)	5 (9.5 mps)	6 (12.5 mps)
0.000	0.00000000	0.00185006	0.00137004	0.00000000	0.00000000	0.00000000
22.500	0.00000000	0.00116003	0.00034001	0.00000000	0.00000000	0.00000000
45.000	0.00000000	0.00151005	0.00007000	0.00000000	0.00000000	0.00000000
67.500	0.00000000	0.00089003	0.00011000	0.00000000	0.00000000	0.00000000
90.000	0.00000000	0.00121004	0.00014000	0.00000000	0.00000000	0.00000000
112.500	0.00000000	0.00187006	0.00050002	0.00000000	0.00000000	0.00000000
135.000	0.00000000	0.00512015	0.00132004	0.00000000	0.00000000	0.00000000
157.500	0.00000000	0.00438013	0.00137004	0.00000000	0.00000000	0.00000000
180.000	0.00000000	0.00523016	0.00215006	0.00000000	0.00000000	0.00000000
202.500	0.00000000	0.00219007	0.00091003	0.00000000	0.00000000	0.00000000
225.000	0.00000000	0.00288009	0.00162005	0.00000000	0.00000000	0.00000000
247.500	0.00000000	0.00139004	0.00247007	0.00000000	0.00000000	0.00000000
270.000	0.00000000	0.00206006	0.00443013	0.00000000	0.00000000	0.00000000
292.500	0.00000000	0.00256008	0.00854026	0.00000000	0.00000000	0.00000000
315.000	0.00000000	0.00457014	0.01142034	0.00000000	0.00000000	0.00000000
337.500	0.00000000	0.00258008	0.00530016	0.00000000	0.00000000	0.00000000

TABLE 6-6 (cont.)

Frequency of Occurrence of Wind Speed, Direction and Stability
Stability Category 6

Direction (degrees)	Wind Speed Category 1 (1.5 mps)	2 (2.5 mps)	3 (4.3 mps)	4 (6.8 mps)	5 (9.5 mps)	6 (12.5 mps)
0.000	0.01015030	0.00331010	0.00000000	0.00000000	0.00000000	0.00000000
22.500	0.00803024	0.00212006	0.00000000	0.00000000	0.00000000	0.00000000
45.000	0.01573047	0.00327010	0.00000000	0.00000000	0.00000000	0.00000000
67.500	0.00701021	0.00126004	0.00000000	0.00000000	0.00000000	0.00000000
90.000	0.01304039	0.00256008	0.00000000	0.00000000	0.00000000	0.00000000
112.500	0.00662020	0.00176005	0.00000000	0.00000000	0.00000000	0.00000000
135.000	0.01879056	0.00459014	0.00000000	0.00000000	0.00000000	0.00000000
157.500	0.01343040	0.00432013	0.00000000	0.00000000	0.00000000	0.00000000
180.000	0.01859056	0.00667020	0.00000000	0.00000000	0.00000000	0.00000000
202.500	0.01022031	0.00290009	0.00000000	0.00000000	0.00000000	0.00000000
225.000	0.01405042	0.00539016	0.00000000	0.00000000	0.00000000	0.00000000
247.500	0.00687021	0.00354011	0.00000000	0.00000000	0.00000000	0.00000000
270.000	0.01136034	0.00443013	0.00000000	0.00000000	0.00000000	0.00000000
292.500	0.01126034	0.00612018	0.00000000	0.00000000	0.00000000	0.00000000
315.000	0.02234067	0.00998030	0.00000000	0.00000000	0.00000000	0.00000000
337.500	0.01206036	0.00518016	0.00000000	0.00000000	0.00000000	0.00000000

TABLE 6-6 (cont.)

Vertical Potential Temperature Gradient (Degrees Kelvin/meter)

	Wind speed category 1	Wind speed category 2	Wind speed category 3	Wind speed category 4	Wind speed category 5	Wind speed category 6
Stab. Category 1	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}
Stab. Category 2	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}
Stab. Category 3	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}
Stab. Category 4	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}	0.000000×10^{0}
Stab. Category 5	0.20000×10^{-1}	0.20000×10^{-1}	0.20000×10^{-1}	0.20000×10^{-1}	0.20000×10^{-1}	0.20000×10^{-1}
Stab. Category 6	0.35000×10^{-1}	0.35000×10^{-1}	0.35000×10^{-1}	0.35000×10^{-1}	0.35000×10^{-1}	0.35000×10^{-1}

TABLE 6-6 (cont.)

Wind Profile Power Law Exponents

	Wind speed category 1	Wind speed category 2	Wind speed category 3	Wind speed category 4	Wind speed category 5	Wind speed category 6
Stab. Category 1	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$
Stab. Category 2	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$	$0.70000\text{x}10^{-1}$
Stab. Category 3	$0.100000\text{x}10^{0}$	$0.100000\text{x}10^{0}$	$0.100000\text{x}10^{0}$	$0.100000\text{x}10^{0}$	$0.100000\text{x}10^{0}$	$0.100000\text{x}10^{0}$
Stab. Category 4	$0.150000\text{x}10^{0}$	$0.150000\text{x}10^{0}$	$0.150000\text{x}10^{0}$	$0.150000\text{x}10^{0}$	$0.150000\text{x}10^{0}$	$0.150000\text{x}10^{0}$
Stab. Category 5	$0.350000\text{x}10^{0}$	$0.350000\text{x}10^{0}$	$0.350000\text{x}10^{0}$	$0.350000\text{x}10^{0}$	$0.350000\text{x}10^{0}$	$0.350000\text{x}10^{0}$
Stab. category 6	$0.550000\text{x}10^{0}$	$0.550000\text{x}10^{0}$	$0.550000\text{x}10^{0}$	$0.550000\text{x}10^{0}$	$0.550000\text{x}10^{0}$	$0.550000\text{x}10^{0}$

$$
\begin{aligned}
N_{max} &= RAC/SRR_{VOL} \\
&= (1.2\times10^{-4}\ mg/m^3)/(4.8\times10^{-8}\ kg/m^3) \\
&= 2500\ mg/kg
\end{aligned}
$$

If dry-weight concentrations of benzene exceed this value, then sludge should not be allowed to accumulate in the lagoons at this site.

6.6.1.2. SAMPLE CALCULATION: TULSA, OKLAHOMA -- The Tulsa, Oklahoma, facility consists of three impoundments connected in series. The impoundments receive sludge in continuous inflow and release decanted effluent for return to the wastewater treatment facility. Sludge is removed from the impoundments about every 9-12 months. The lagoons are judged to be poorly mixed, with the concentrations of sludge varying across the impoundment length. For these sample calculations, the individual impoundments contained in the Tulsa facility will therefore be modeled as a single aggregate impoundment. Table 6-7 contains input parameters appropriate for modeling the Tulsa site, as well as results from the calculations described below. As before, a reference air concentration (RAC) of 1.2×10^{-4} is selected, corresponding to a risk level of 10^{-6}.

The expected ratio between the concentration of dissolved benzene in sludge received by the first lagoon and the dry-weight concentration of benzene in the sludge is calculated from Equation 4-1, where the ratio of solids to liquids in that sludge is reported to be about 0.016 kg/*l*:

$$C_0/N = 1/(k_d + r_{sl}^{-1}) = 1/[37+(0.016)^{-1}] = 0.01\ kg/l = 10\ kg/m^3$$

Expected emissions of benzene are next related to the concentration of dissolved contaminant in sludge received by the first lagoon, based on equations developed in Section 6.3.7.2 for poorly mixed systems. As before, the overall mass transfer coefficient (k) is estimated from the liquid and gas phase mass transfer coefficients (k_g and k_l). With a total area of 61,000 m^2, the lagoons in Tulsa have an effective diameter, or fetch, of:

TABLE 6-7

Derivation of Criteria for the Air Pathway
Tulsa, Oklahoma

Parameter	Value	(Units)
A	61,000	(m^2)
h	3.1	(m)
V	$1.9x10^5$	(m^3
Q	0.01	(m^3/sec)
r_{sl}	16	(kg/m^3)
k_{sl}	0.59	(unitless)
U_{10}	4.9	(m/sec)
T	298	(°K)
d_e	280	(m)
U^*	0.15	(m/sec)
k_l	$6.9x10^{-6}$	(m/sec)
k_g	$2.3x10^{-3}$	(m/sec)
k	$6.8x10^{-6}$	(m/sec)
C_0/N	10	(kg/m^3)
E_{site}/C_0	$7.1x10^{-2}$	(m^3/sec)
C_a/E_{site}	$6.5x10^{-6}$	(sec/m^3)
SRR_{vol}	$4.6x10^{-7}$	(kg/m^3)
N_{max}	260	(mg/kg)

$$d_e = 2(A/\pi)^{0.5} = 2(61{,}000/3.142)^{0.5} = 280 \text{ m}$$

Since the depth of the lagoons is reported to be about 3.1 meters, the fetch:depth ratio is (280/3.1) = 90 m. Average wind speed in Tulsa (4.9 m/sec) is greater than 3.2 m/sec, and the fetch:depth ratio (93) is greater than 52, so k_l (in m/sec) can be calculated from Equation 5-23.

$$k_l = 2.611\text{x}10^{-7}\, U_{10}^{2}\, [D_{cw}/D_{eth}]^{(2/3)} = 2.611\text{x}10^{-7}\, (4.9)^2\, [9.8\text{x}10^{-6})/(8.5\text{x}10^{-6})]^{(2/3)}) = 6.9\text{x}10^{-6}$$

The mass transfer coefficient for the gas side is estimated from Equation 5-24:

$$k_g = 1.8\text{x}10^{-3}\, U_w^{0.78}\, Sc_g^{-0.67}\, d_e^{-0.11} = (1.8\text{x}10^{-3})(4.9)^{0.78}(1.70)^{-0.67}(279)^{-0.11} = 2.3\text{x}10^{-3} \text{ m/sec}$$

These estimates of k_l and k_g are substituted into Equation 6-5:

$$1/k = 1/(6.9\text{x}10^{-6}) + (8.21\text{x}10^{-5})(298)/[(5.5\text{x}10^{-3})(2.3\text{x}10^{-3})]$$

or

$$k = 6.8\text{x}10^{-6} \text{ m/sec}$$

Equation 6-17 for plug flow systems gives the ratio of expected benzene emissions to the dissolved concentration of benzene in liquid received by the lagoons. First, α must be calculated:

$$\begin{aligned}\alpha &= \frac{k\,A + k_b\,V\,(1+k_{sl})}{Q(1+k_{sl})}\\ &= \frac{(6.8\text{x}10^{-6})(61{,}000)+(1.7\text{x}10^{-6})(190{,}000)(1.59)}{(0.01)(1.59)}\\ &= 58\end{aligned}$$

Substituting this value into Equation 6-17 yields:

$$E_{site}/C_0 = k\ A\ [1-\exp(-\alpha)]\ /\ \alpha = (6.8\text{x}10^{-6})(61{,}000)[1-\exp(-32.5)]/(32.5) = 7.1\text{x}10^{-3}$$

To estimate the expected concentration of benzene in ambient air as function of emissions from the Tulsa site, the ISCLT model is executed within GEMS, based on STAR data from the Tulsa airport (latitude 36.2, longitude 95.9). Results from ISCLT execution reveal that for a unit emission rate of 1 g/m^2-sec from the lagoons, the maximum expected concentration in ambient air beyond the property boundary is 0.40 g/m^3, so C_a/E_{site} can be calculated as:

$$C_a/E_{site} = (0.40\ \text{g/m}^3)/(61{,}000\ \text{g/sec}) = 6.5\text{x}10^{-6}\ \text{sec/m}^3$$

The source-receptor ratio is next calculated by combining the results derived above:

$$\begin{aligned} SRR_{VOL} &= (C_a/E_{site})(E_{site}/C_0)(C_0/N) \\ &= (6.5\text{x}10^{-6}\ \text{sec/m}^3)(7.1\text{x}10^{-2}\ \text{m}^3\text{/sec})(10\ \text{kg/m}^3) \\ &= 4.6\text{x}10^{-7}\ \text{kg/m}^3 \end{aligned}$$

From the source-receptor ratio can be derived the maximum allowable dry-weight concentration of benzene in sludge received by the lagoons in Tulsa:

$$N_{max} = RAC/SRR_{VOL} = (1.2\text{x}10^{-4}\ \text{mg/m}^3)/(4.6\text{x}10^{-7}\ \text{kg/m}^3) = 260\ \text{mg/kg}$$

6.6.1.3. SAMPLE CALCULATION: PORTLAND, OREGON -- The third sample calculation involves an impoundment for permanent disposal of sludge. The Portland facility uses its large (13 ha) impoundment only occasionally, when the primary means of sludge reuse (composting) is inoperative. Such failures occur infrequently. For the sample calculations presented here, it will be assumed that the composting operation fails once each year, and that upon failure of that system, the impoundment receives 260,000 gallons of sludge, with 1.8% solids content. The calculations will be based on the conservative assumption that each sludge deposit spreads to a thin layer that covers the entire surface of the impoundment. Because deposits to the impoundment are sporadic and relatively infrequent, the facility will be modeled with methods presented in Section 6.3.7.3. Input parameters for the calculations are provided by Table 6-8. As before the calculations will be based on a reference air concentration of 1.2×10^{-4} mg/m^3.

Based on the reported ratio of solids to liquids (1.8%) in each deposit of sludge, the ratio of the concentration of dissolved benzene to the dry-weight concentration of benzene received by the lagoon is estimated from Equation 4-1:

$$C_0/N = 1/(k_d + r_{sl}^{-1}) = 1/[37+(0.018)^{-1}] = 1.1 \times 10^{-2} \text{ kg}/l = 11 \text{ kg/m}^3$$

Expected emissions of benzene are next related to the concentration of dissolved contaminant in sludge deposited in the first impoundment, based on Equation 6-18 for impoundments with infrequent deposits of sludge. First, the overall mass transfer coefficient (k) is estimated from the liquid and gas phase mass transfer coefficients (k_g and k_ℓ). Based on the assumption that the sludge spreads to cover the entire surface of the impoundment (with a total area of 130,000 m^3), the effective diameter, or "fetch" of the surface is:

$$d_e = 2(A/\pi)^{0.5} = 2(130{,}000/3.142)^{0.5} = 410 \text{ m}$$

The depth of such a layer of sludge would be only 7.7×10^{-3} m, so that the fetch:depth ratio is much greater than 51.2. Average wind speed in Portland (3.7 m/sec) is greater than 3.2 m/sec, so k_ℓ (in m/sec) can be calculated from Equation 5-23.

$$k_\ell = 2.611 \times 10^{-7} \, U_{10}^2 \, [D_{cw}/D_{eth}]^{(2/3)} = 2.611 \times 10^{-7} \, (3.7)^2 \, [9.8 \times 10^{-6})/(8.5 \times 10^{-6})]^{(2/3)} = 3.9 \times 10^{-6}$$

The mass transfer coefficient for the gas side is estimated from Equation 5-24:

TABLE 6-8

Derivation of Criteria for the Air Pathway
Portland, Oregon

Parameter	Value	(Units)
A	130,000	(m^2)
V_{dep}	1000	(m^3)
r_{sl}	18	(kg/m^3)
k_{sl}	0.67	(*l*/kg)
U_{10}	3.7	(m/sec)
T	298	(°K)
d_e	410	(m)
U^*	1.1×10^{-1}	(m/sec)
k_l	3.9×10^{-6}	(m/sec)
k_g	1.8×10^{-3}	(m/sec)
k	3.9×10^{-6}	(m/sec)
C_0/N	11	(kg/m^3)
E_{site}/C_0	5.3×10^{-5}	(m^3/sec)
C_a/E_{site}	3.4×10^{-6}	(sec/m^3)
SRR_{vol}	1.9×10^{-9}	(kg/m^3)
N_{max}	63,000	(mg/kg)

$$k_g = 1.8\text{x}10^{-3}\, U_w^{0.78}\, Sc_g^{-0.67}\, d_e^{-0.11} = (1.8\text{x}10^{-3})(3.7)^{0.78}(1.70)^{-0.67}(410)^{-0.11} = 1.8\text{x}10^{-3} \text{ m/sec}$$

These estimates of k_l and k_g are then substituted into Equation 6-5:

$$1/k = 1/(3.9\text{x}10^{-6}) + (8.21\text{x}10^{-5})(298)/[(5.5\text{x}10^{-3})(1.8\text{x}10^{-3})]$$

or

$$k = 3.9\text{x}10^{-6} \text{ m/sec}$$

Equation 6-18 gives the ratio of expected benzene emissions to the dissolved concentration of benzene in liquid received by the lagoons, but requires the calculation of a value for β:

$$\beta = \frac{(k\ A\ V_{dep}^{-1}) + k_b(1+k_{sl})}{1+k_{sl}}$$

$$\beta = \frac{(3.9\text{x}10^{-6})(130{,}000)(1000)^{-1} + 1.7\text{x}10^{-6}(1.67)}{1.67} = 3.1\text{x}10^{-4}$$

Substituting this value into Equation 6-18 gives:

$$\begin{aligned} E_{site}/C_0 &= \frac{kA\ [1-\exp(-\beta T)]}{\beta T} \\ &= \frac{(3.9\text{x}10^{-6})(1.3\text{x}10^{5})(1-\exp[(-3.1\text{x}10^{-4})(3.2\text{x}10^{7})])}{(3.1\text{x}10^{-4})(3.2\text{x}10^{7})} \\ &= 5.3\text{x}10^{-5} \text{ m}^3\text{/sec} \end{aligned}$$

To estimate the expected concentration of benzene in ambient air as function of emissions from the Portland site, the ISCLT model is executed within GEMS, based on STAR data retrieved from the Portland airport, for latitude 45.6 and longitude 122.6. Results from ISCLT execution reveal that for a unit emission rate of 1 g/m^2-sec from the lagoons, the maximum expected concentration in ambient air beyond the property boundary is 0.44 g/m^3. C_a/E_{site} can then be calculated as:

$$C_a/E_{site} = (0.44\ g/m^3)/(130{,}000\ g/sec) = 3.4\times10^{-6}\ sec/m^3$$

The source-receptor ratio is calculated by combining previous results (as listed in Table 6-8):

$$\begin{aligned} SRR_{VOL} &= (C_a/E_{site})(E_{site}/C_0)(C_0/N) \\ &= (3.4\times10^{-6}\ sec/m^3)(5.3\times10^{-5}\ m^3/sec)(11\ kg/m^3) \\ &= 1.9\times10^{-9}\ kg/m^3 \end{aligned}$$

This ratio is used to calculate the maximum allowable dry-weight concentration for benzene in sludge deposited in the lagoon:

$$N_{max} = RAC/SRR_{VOL} = (1.2\times10^{-4}\ mg/m^3)/(2.0\times10^{-9}\ kg/m^3) = 60{,}000\ mg/kg$$

6.6.2. Analysis of Exposure for the Most Exposed Populations.

6.6.2.1. SAMPLE CALCULATION: ANTRIM, NEW HAMPSHIRE -- Tier 2 calculations provide an upper bound estimate of maximum concentrations likely to be encountered at a specified receptor location. Conversely, as illustrated above, they provide a derivation of maximum allowable sludge concentrations, based on specified reference air concentrations, site characteristics and a specified receptor distance.

The distribution of potential exposure over the entire exposed population may also be of interest. Execution of the complete ISCLT model can provide a more complete picture of the distribution of potential exposure over the total exposed population (and a determination of how criteria would vary with alternative definitions of the MEI). To derive such a distribution for the Antrim facility, the ISCLT model was executed with meteorological data from the Concord, New Hampshire, airport STAR station, and with census data for each area within 50 kilometers of Antrim, New Hampshire. The model reported both expected concentrations and sizes of exposed populations for each of 30 ring distances in each of 16 radial directions, for a total of 480 locations.

Figure 6-1 shows the estimated distribution of population surrounding the Antrim facility. As can be seen from the figure, approximately 450,000 persons reside within 50 kilometers of the facility; of these, over 95% reside at a distance greater than 10 kilometers. Approximately 1,000 people live within 1 kilometer of the Antrim facility. Figure 6-2 shows estimated ground-level concentrations of contaminant (per unit of emissions from the site) for each of 4 radial directions selected from the 16 examined by the model. Most of the contamination occurs within 500 meters of the facility with similar concentration reduction patterns for the four radial directions. As can be seen from a simple comparison of these two graphs, most of the exposed population will be exposed to ambient air concentrations considerably lower than those within the innermost ring distances. Figure 6-3 combines estimated air concentration with population data to derive a distribution of air concentrations as a function of the number of persons exposed. Based on a unit rate of emissions from the Antrim site (1 g/sec), the graph presents the number of persons expected to breathe air at each level of contamination. From the graph, for example, it can be seen that approximately 100 persons are expected to be exposed to air with at least 0.04 mg/m^3 of contaminant for each g/sec of emissions from the Antrim facility. Approximately 30 people are estimated to be exposed to the relatively high contaminant concentration of 0.08 mg/m^3 per g/m^2-sec of emission from the Antrim facility.

If:

$$E_{site}/N = (E_{site}/C_0)(C_0/N) = (9.9\times10^{-3}\ m^3/sec)(0.53\ kg/m^3) = 5.3\times10^{-3}\ kg/sec$$

then 5.3×10^{-3} mg/sec (or 5.3×10^{-6} g/sec) of benzene is expected to be emitted from the lagoons for each mg/kg dry-weight of benzene in influent. Combining this result with data presented in Figure 6-3 shows that for each mg/kg of contaminant concentration in Antrim sludge, about 30 persons are expected to be exposed to more than 4×10^{-7} mg/m^3 of contaminant concentration in ambient air.

6.6.2.2. SAMPLE CALCULATION: TULSA, OKLAHOMA -- Similar distributions of populations by distance and exposure level have been derived for the area surrounding the Tulsa facility. Calculations are based on weather and census data for the area within 50 kilometers of the

FIGURE 6-1

Cumulative Distribution of Population

By Distance from Antrim Facility

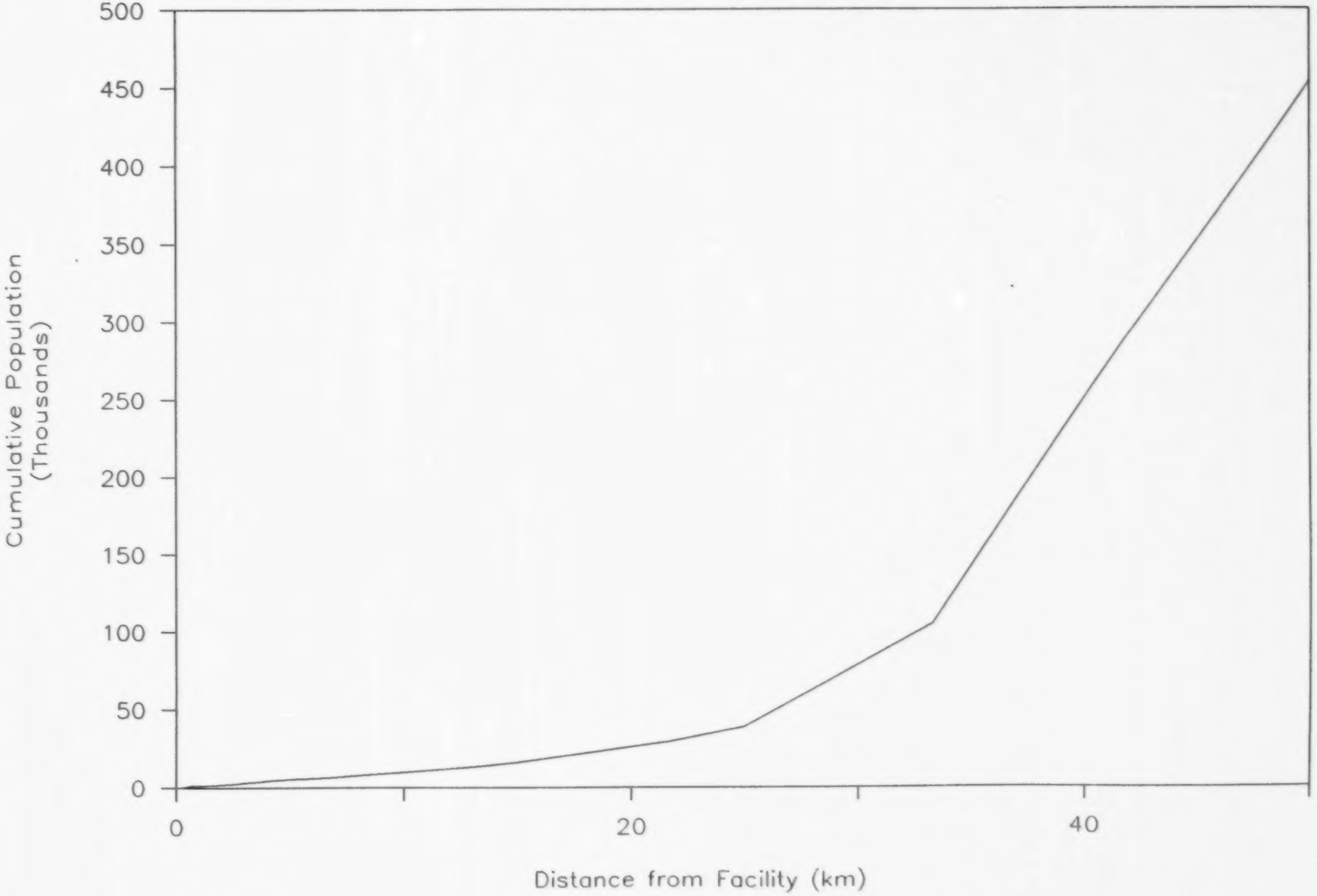


FIGURE 6-2

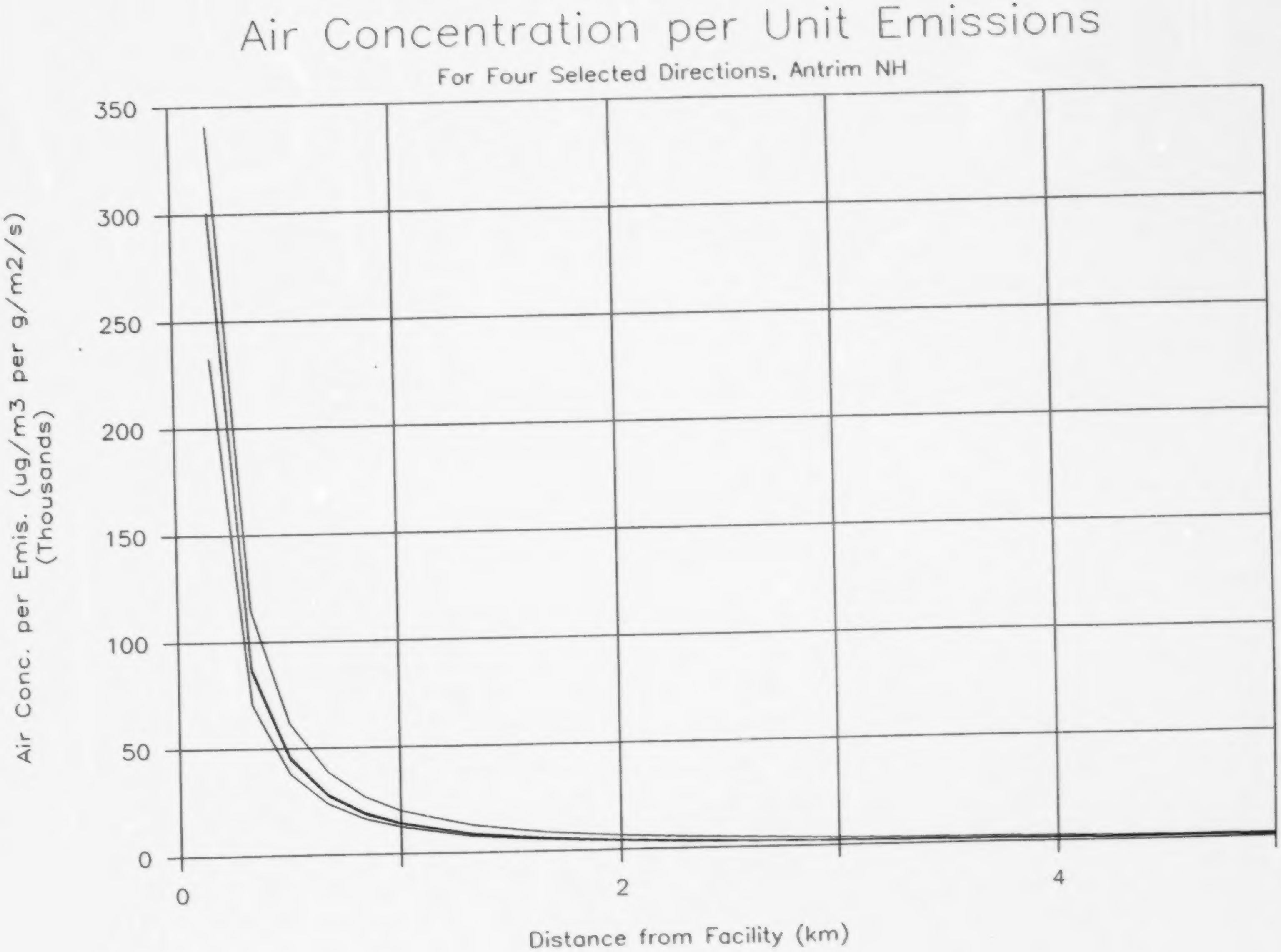
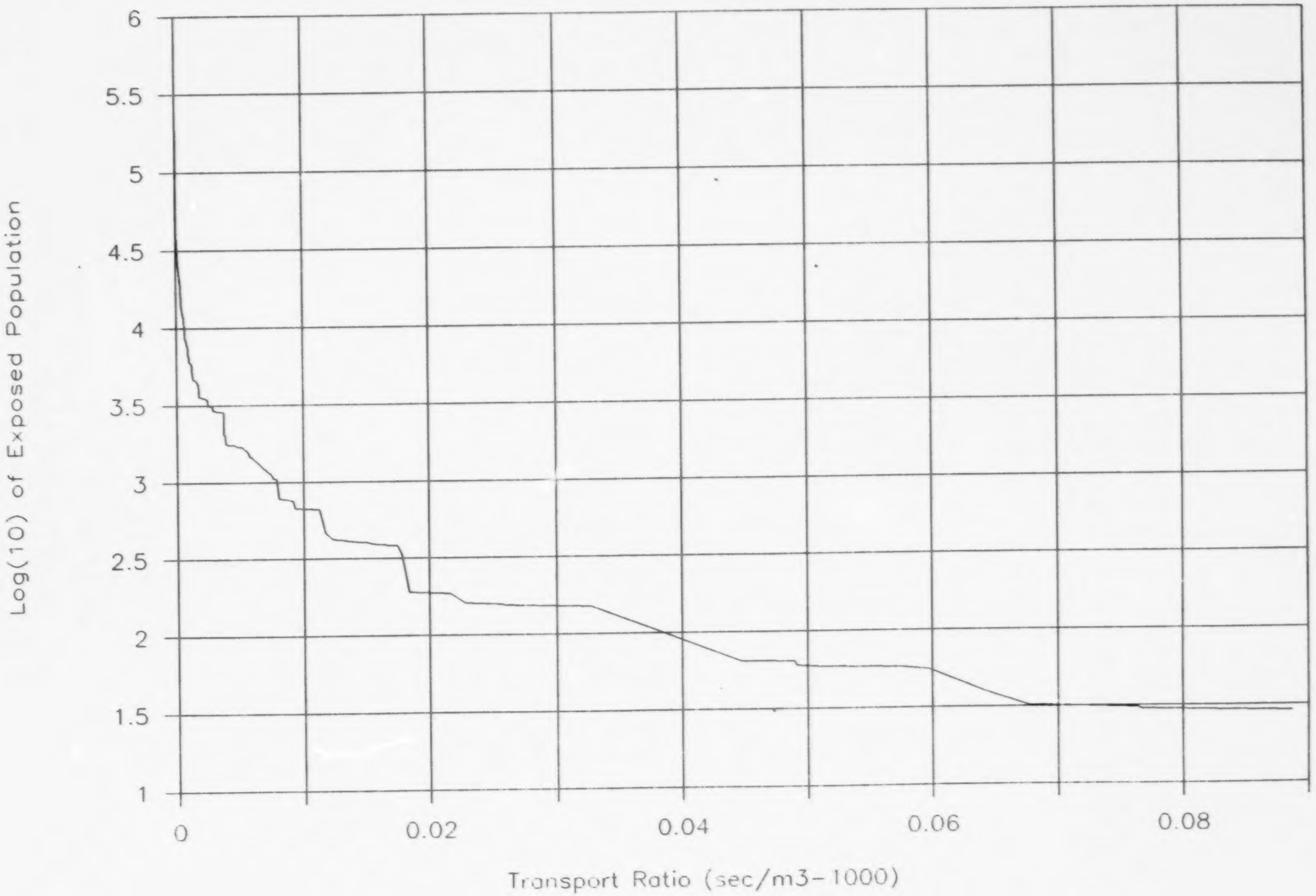

Air Concentration per Unit Emissions
For Four Selected Directions, Antrim NH
Air Conc. per Emis. (ug/m3 per g/m2/s)
(Thousands)
350
300
250
200
150
100
50
0
0
2
4
Distance from Facility (km)

FIGURE 6-3

Transport Ratio by Population

Antrim, New Hampshire



center of Tulsa. Figure 6-4 shows the distribution of populations by distance from the Tulsa facility. As shown by the graph, approximately 600,000 persons live within 50 kilometers of the facility. In contrast to Antrim, about half of this population lives within 10 kilometers of the facility; about 10,000 people live within 1 kilometer of the facility.

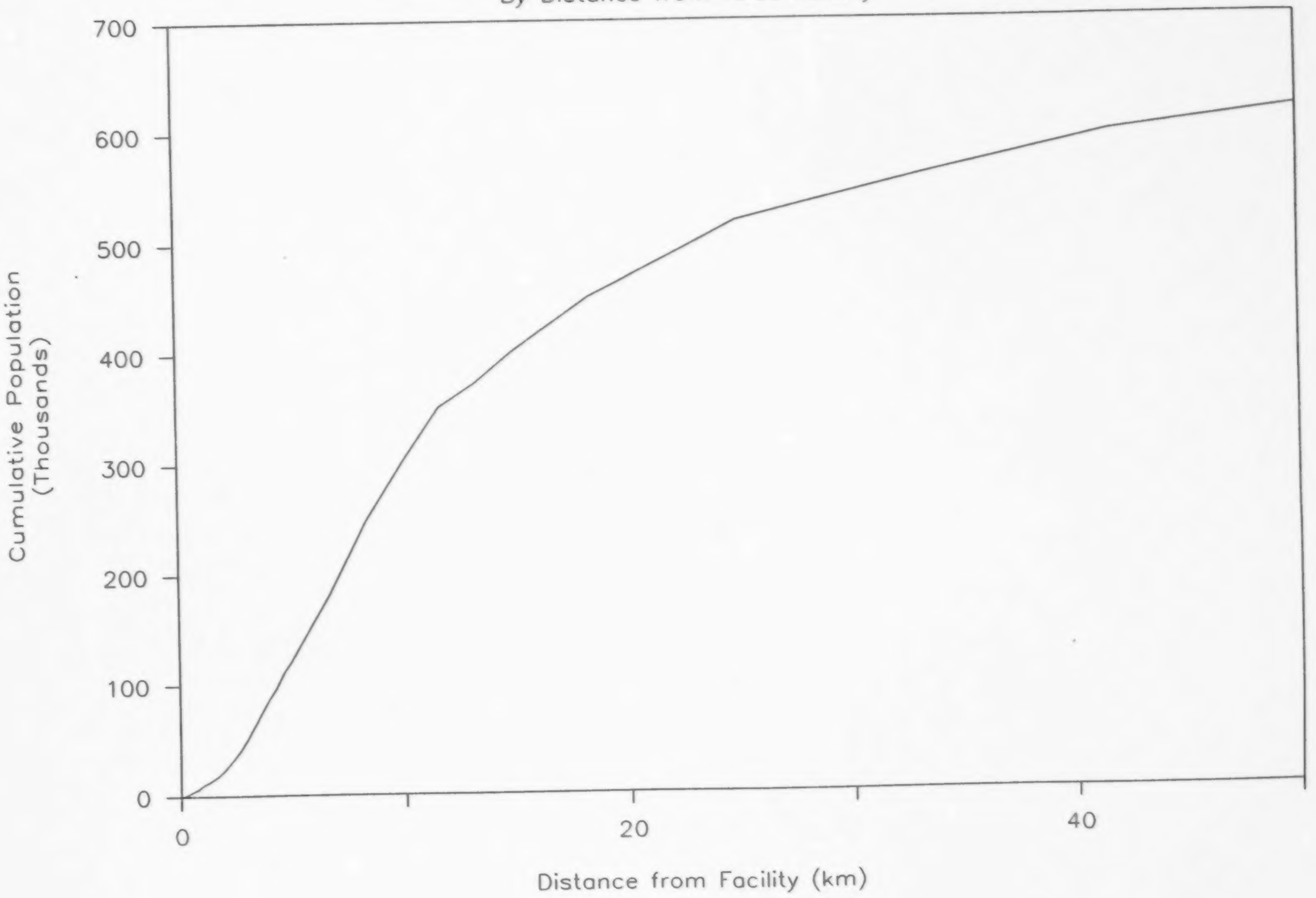
Ambient air concentrations per unit emissions are shown in Figure 6-5 for four radial directions. The air concentrations in Tulsa are generally higher than the air concentrations in Antrim, and fall sharply within the first 500 meters. As before, the estimate of population distribution has been combined with the estimates of expected ambient air concentrations per unit emission to derive a distribution of the number of persons exposed as a function of air concentrations. Results are provided by Figure 6.6. As can be seen from the graph, a larger population (about 100,000 persons) is potentially exposed in the Tulsa area than in the Antrim area, but the highest level of contamination per unit emissions is lower in the area surrounding Tulsa than the area around Antrim.

Results from Section 6.6.1.2 can be combined to show that $7.2\text{x}10^{-5}$ g/sec of benzene are expected to be emitted from Tulsa impoundments for each mg/kg dry-weight of benzene in the sludge. It can therefore be shown from Figure 6-6 that about 3 people would be exposed to a concentration greater than $2\text{x}10^{-6}$ mg/m^3 per mg/kg concentration in sludge.

6.6.2.3. SAMPLE CALCULATION: PORTLAND, OREGON -- Calculations described in Sections 6.6.2.1 and 6.6.2.2 were repeated for the Portland facility, based on meteorological data from the STAR station at Portland and census data for persons living within 50 kilometers of the center of Portland. The estimated distribution of population by distance from the facility is presented in Figure 6-7, indicating a greater population density (about 1.3 million people) in the 50-kilometer radius surrounding the Portland area than in the area surrounding Antrim or Tulsa. About 9,000 people live within 1 kilometer of the location modeled.

Attenuation of air concentrations in four selected directions is graphed in Figure 6-8. The air concentrations fall substantially by a distance of 1 kilometer from the facility. Figure 6-9 provides a graph of the size of exposed populations by the ratio of ambient air concentrations per unit of emissions from the Portland facility. Although the Portland population potentially exposed

FIGURE 6-4



Cumulative Distribution of Population

By Distance from Tulsa Facility

FIGURE 6-5

Air Concentration per Unit Emissions

For Four Selected Directions, Tulsa OK

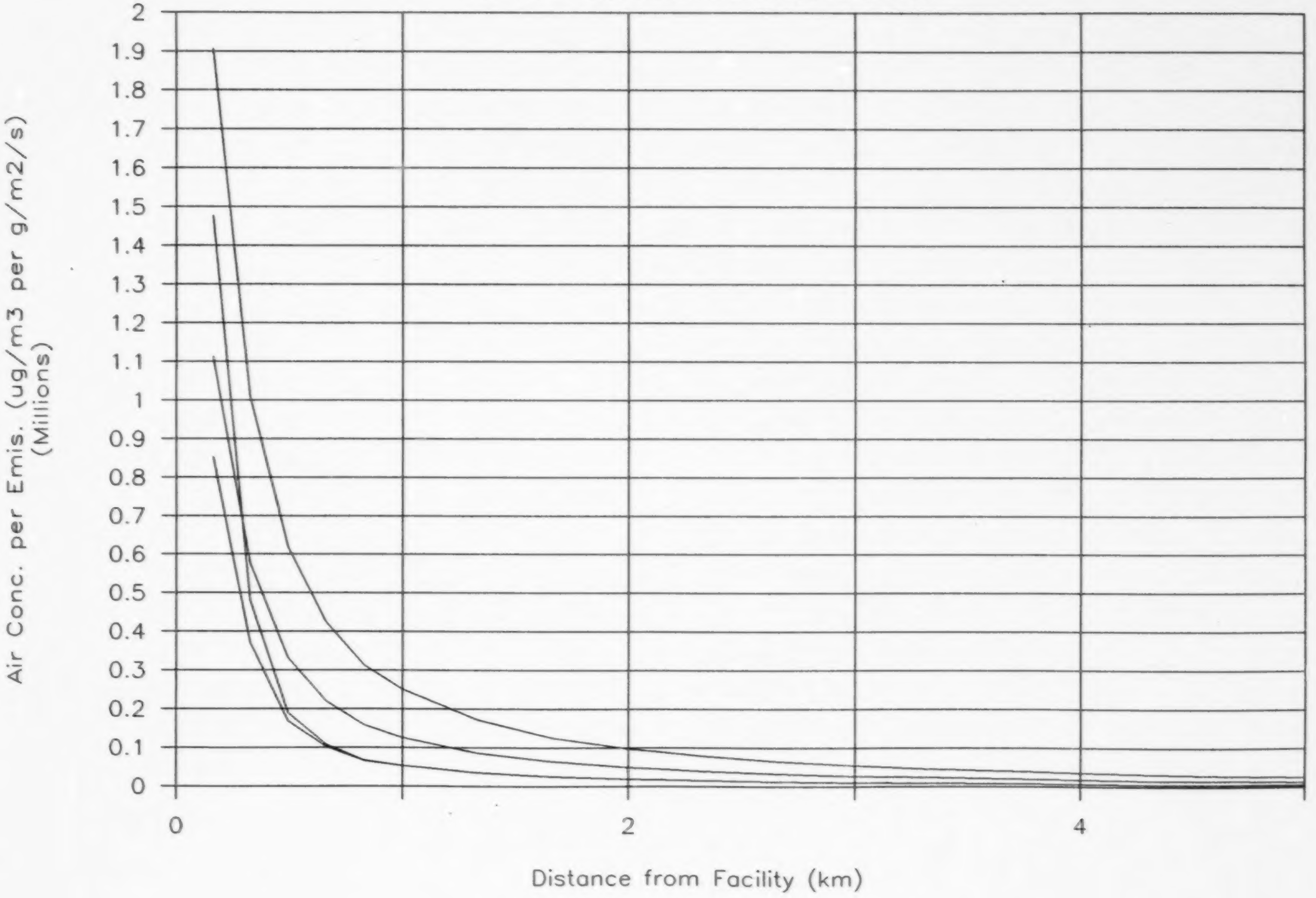


FIGURE 6-6

Transport Ratio by Population

Tulsa, Oklahoma

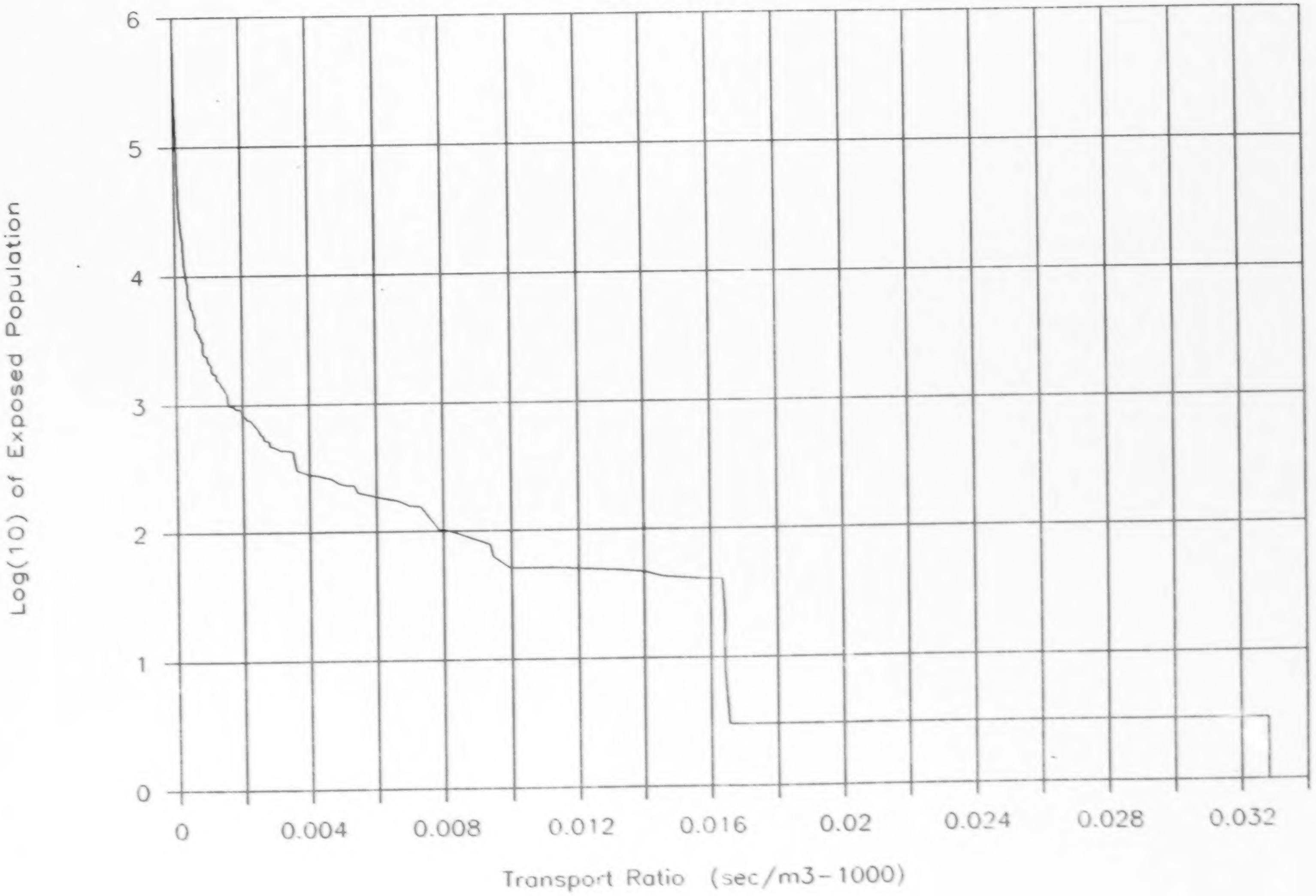


FIGURE 6-7

Cumulative Distribution of Population

By Distance from Portland Facility

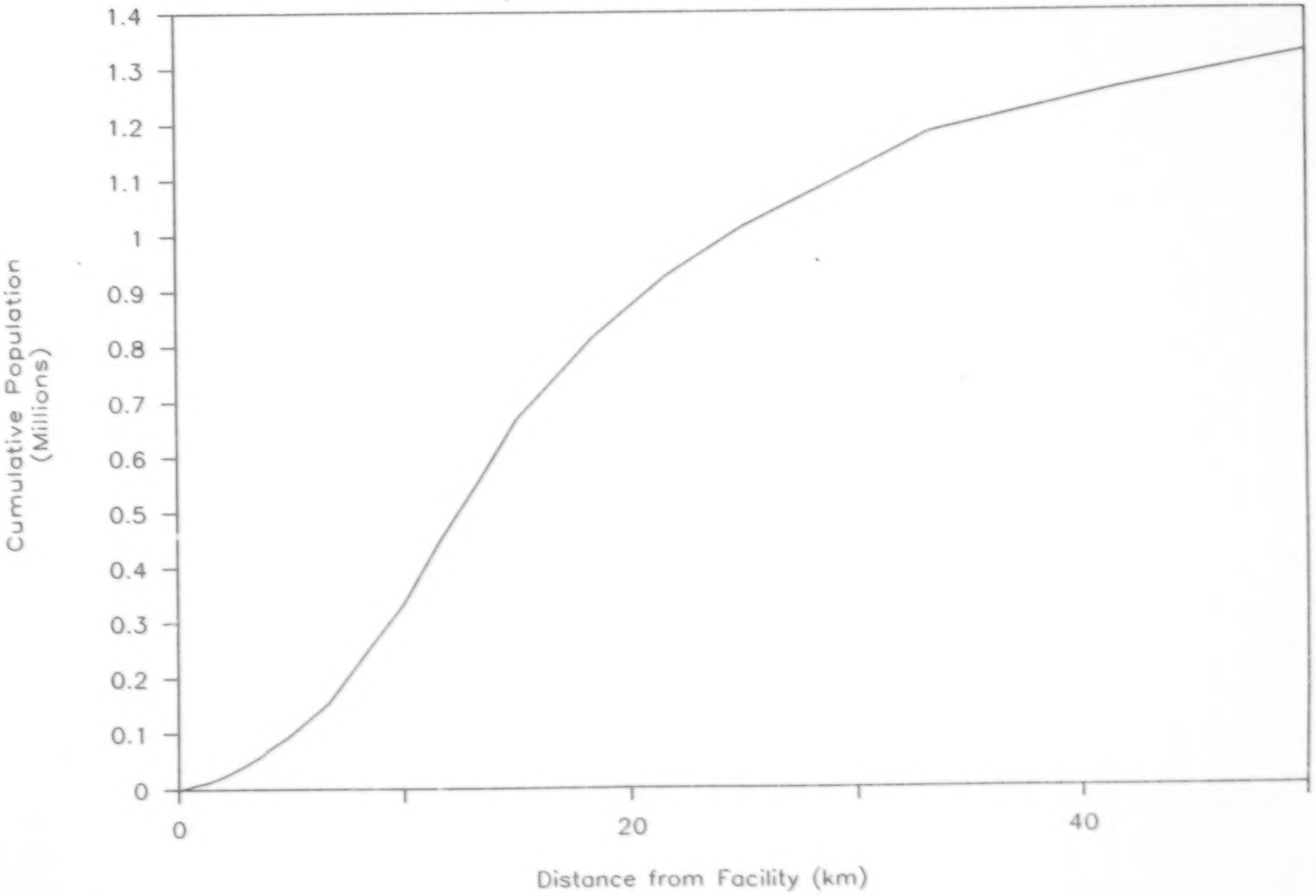


FIGURE 6-8

Air Concentration per Unit Emissions

For Four Selected Direct., Portland OR

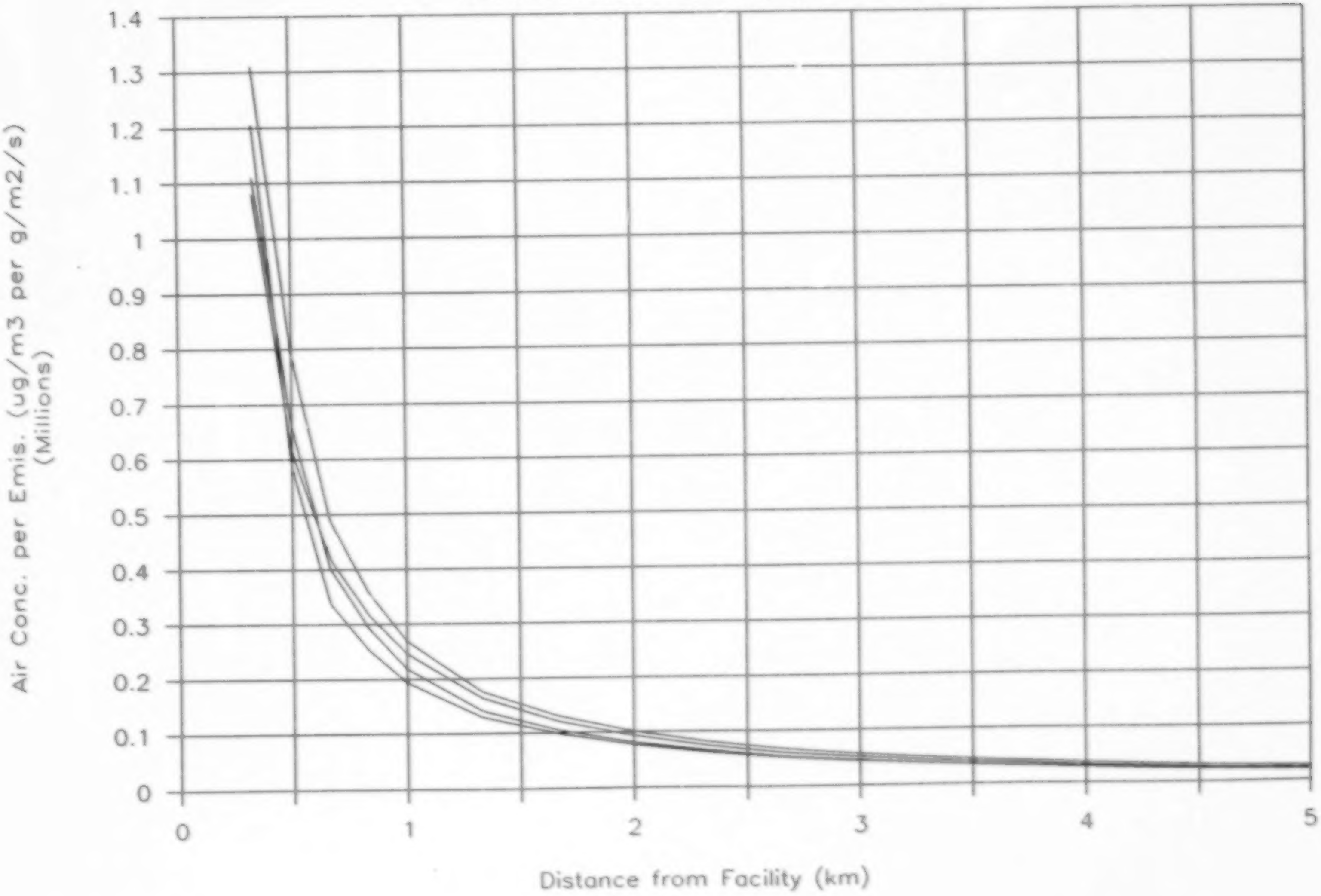
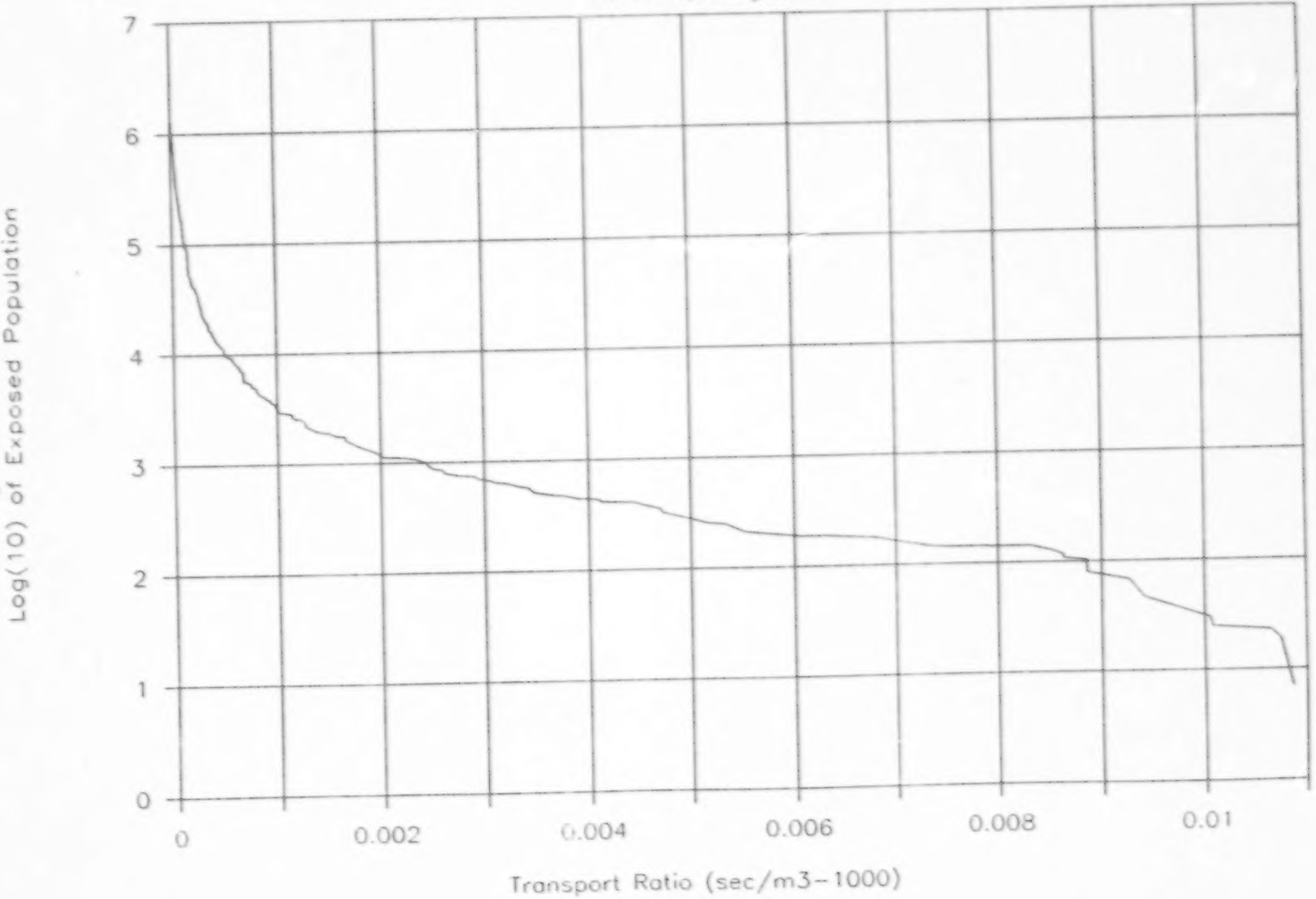


FIGURE 6-9

Transport Ratio by Population

Portland, Oregon



is about 1 million people, the concentration to which the MEI is exposed per unit of emissions is lower than was described above for Tulsa and Antrim. If 5.7×10^{-7} g/sec are emitted per mg/kg of contaminant in sludge disposed, then about 30 people are exposed to over 6×10^{-9} mg/m^3 per mg/kg of contaminant in sludge disposed in the Portland area.

7.0 SIMULTANEOUS CONSIDERATION OF MULTIPLE PATHWAYS OF EXPOSURE

Chapters 4, 5, and 6 described methods for estimating potential exposure to a most exposed individual (MEI) or to the most exposed populations (MEP). These methods consider each pathway of potential exposure independently; they do not consider the possibility that a single individual could be the most exposed individual for more than one pathway at a time, or even that exposure through one pathway might add to the exposure from another. While it is probably unlikely that a single individual could be the MEI through drinking water pathways associated with both groundwater and surface water, a single individual living near a surface disposal facility might conceivably be the MEI for both groundwater and volatilization pathways, and perhaps also for the fish ingestion pathway associated with surface water contamination. Similarly, significant simultaneous exposure is possible through the air pathway and the drinking water component of the surface water pathway. This chapter briefly outlines an approach for joint consideration of these pathways.

Common to Chapters 4, 5, and 6 was the derivation of source-receptor ratios (SRR_{GW}, SRR_{SW}, and SRR_{VOL}, respectively) that relate the concentration of each contaminant in the respective environmental medium to the dry-weight concentration of contaminant in sludge received by the surface disposal facility. With equations adapted from Sections 4.5, 5.5, and 6.5, these three ratios can be used to predict total exposure to a single MEI exposed through more than one pathway simultaneously. For example, simultaneous exposure through volatilization and groundwater pathways might be described as:

$$HE_{TOT} = N_i \left[\frac{SRR_{VOL} I_a RE_A + SRR_{GW} I_W RE_W + TBI}{BW} \right]$$

where:

HE_{TOT} = total exposure of the MEI to contaminant released from the surface disposal facility (mg/kg-day)
N_i = dry weight concentration of contaminant in sludge (mg/kg)
SRR_{VOL} = source receptor ratio for volatilization pathway: the ratio of the air concentration at the receptor location to the dry weight concentration of the same contaminant in sludge received by the facility (kg/m^3)
I_a = quantity of air inhaled per day (m^3/day)
RE_A = relative effectiveness of exposure to inhaled contaminant (unitless)

SRR_{GW} = source receptor ratio for groundwater pathway: the ratio of the concentration of the contaminant in well water ingested by the MEI to the dry weight concentration of sludge received by the facility (kg/*l*)

I_W = quantity of water ingested per day (*l*/day)

RE_W = relative effectiveness of exposure to ingested contaminant (unitless)

TBI = total exposure to this contaminant from all other sources (mg/kg-day)

BW = body weight of MEI (kg)

If a maximum allowable level of human exposure can be established from the human cancer potency (q_1^*) or risk reference dose (RfD) for the particular contaminant of concern, then the above equation can be used to calculate the maximum dry weight concentration of the contaminant in sludge received or accumulated by the facility.:

$$N_{max} = HE_{max} \left[\frac{BW}{SRR_{VOL} I_a RE_A + SRR_{GW} I_W RE_W + TBI} \right]$$

where:

N_{max} = maximum allowable dry weight concentration of this contaminant in sludge received or accumulated by this surface disposal facility (mg/kg)

HE_{max} = maximum allowable exposure to this contaminant, derived with methods adapted from Sections 4.5, 5.5, and 6.5.

For non-human exposure (i.e., comparison of surface water quality to water quality criteria), the methods described in Chapter 5 could be applied exactly as described in that Chapter. An integrated model code could then identify the limiting pathway or group of pathways for each contaminant, and report the maximum allowable concentration of each contaminant to be accepted by the facility.

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